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STRUCTURAL EVALUATION OF EISENHOWER AND SNELL LOCKS, SAINT LAWRENCE SEAWAY, MASSENA, NEW YORK

(2)

by

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The Eisenhower and Snell Locks were constructed between 1955 and 1958 as part of an international cooperative effort to build the Saint Lawrence Seaway. The US portion of the project was authorized by the Wiley-Dondero Act of 13 May 1954. This act also created the Saint Lawrence Seaway Development Corporation (SLSDC) to construct, operate, and maintain the locks. SLSDC contracted with the Corps of Engineers (CE) to design and construct these locks. The CE has had a long history of providing engineering and review assistance to the SLSDC for the two locks. The SLSDC requested the CE through an Intergovernmental Agency Agreement to preform a structural evaluation study of the two locks to determine the adequacy of the existing locks considering present conditions and future needs, and to determine the advisability of their rehabilitation. The study focuses on the internal structural integrity of the chamber wall monoliths at each lock.

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After an initial visual inspection of the locks, approximately 750 lin ft of 6-in.-diam cores were taken by the Mobile District for the laboratory testing program to support the static and seismic analyses. Portions of the cores were subjected to physical testing to develop parameters for the static and seismic analyses. The physical testing for the concrete consisted of tests to find unconfined compressive strength, Poisson's ratio, elastic modulus, splitting tensile strength, specific gravity, absorption, voids, and ultrasonic pulse velocity. Petrographic examinations of three specimens were conducted on the in-place concrete from Eisenhower and Snell Locks.

Three typical concrete chamber wall monoliths were analyzed: a north and a south wall at Eisenhower and a north wall at Snell. Each monolith was analyzed for three loading conditions: normal operation with upper pool in the chamber, normal operation with lower pool in the chamber, and the maintenance condition with the chamber fully dewatered.

For static loading and with the current condition as analyzed, the chamber wall monoliths at Eisenhower and Snell Locks show no evidence of being overstressed. The results of the static analyses show that the chamber wall monoliths have tensile stresses below 200 psi and compressive stresses less than 1,000 psi. These stress levels are below any of the strengths determined from the laboratory testing program (390 psi for tensile splitting tests and 3,660 psi for compression tests).

A response spectra seismic analysis was performed for a north chamber wall monolith at Eisenhower. The response spectra (Operating Basis Earthquake (OBE)) and Maximum Credible Earthquake (MCE)) developed from the geological-seismological investigation were used to perform the linear dynamic stress analysis. For both the OBE and MCE conditions, catastrophic failure of the lock wall is unlikely to occur. No damage should occur during the OBE event with a maximum principal stress of 300 psi. There is a possibility of damage to the structure during the MCE event near the top of the structure where the sloping face transitions into a vertical face. If damage does occur during the MCE event, no more than about 5 to 10 percent of the section would be damaged.

14. (Concluded).

Concrete testing
Eisenhower Lock
Finite element analysis
Seismic-stress analysis
Snell Lock
Soil-structure interaction
Static-stress analysis
Structural evaluation

EXECUTIVE SUMMARY

The Eisenhower and Snell Locks were constructed between 1955 and 1958 as part of an international cooperative effort to build the Saint Lawrence Seaway. In February 1990, the Saint Lawrence Seaway Development Corporation (SLSDC) requested the Corps of Engineers (CE), through an Intergovernmental Agency Agreement, to perform a structural evaluation study of these two locks to determine the adequacy of the existing locks considering present conditions and future needs, and to determine the advisability of their rehabilitation. The study focuses on the internal structural integrity of the chamber wall monoliths at each lock.

The study consisted of three phases:

a. Phase I - Field Investigation and Laboratory Testing Program: This phase of the study explored the current state of the in situ concrete in the chamber wall monoliths and determined the concrete properties needed for the stress analyses in the subsequent phases of the study.

b. Phase II - Static Stress Analysis: In this phase of the study, the state of stress within the chamber wall monoliths is examined by performing nonlinear, two-dimensional, soil-structure interaction finite element analyses of two monoliths at Eisenhower Lock and one monolith at Snell Lock.

c. Phase II - Seismic Stress Analysis: Dynamic finite element analyses of two monoliths are performed to assess their structural integrity during an earthquake. To support the dynamic analysis, a geological-seismological investigation was conducted to provide the ground motions, time histories, and corresponding response spectra that could be expected during a seismic event in the region.

After an initial visual inspection of the locks, approximately 750 lin ft of 6-in.-diam cores were taken for the laboratory testing program to support the static and seismic analyses. Portions of the cores were subjected to physical testing to develop parameters for the static and seismic analyses. The physical testing for the concrete consisted of unconfined compressive strength, Poisson's ratio, elastic modulus, splitting tensile strength, specific gravity, absorption, voids, and ultrasonic pulse velocity. Petrographic examinations of three specimens were conducted on the in-place concrete from Eisenhower and Snell Locks.

Three typical concrete chamber wall monoliths were analyzed: a north and a south wall at Eisenhower and a north wall at Snell. Each monolith was analyzed for

three loading conditions: normal operation with upper pool in the chamber, normal operation with lower pool in the chamber, and the maintenance condition with the chamber fully dewatered. For static loading and with the current condition as analyzed, the chamber wall monoliths at Eisenhower and Snell Locks show no evidence of being overstressed. The results of the static analyses show that the chamber wall monoliths have tensile stresses below 200 psi and compressive stresses less than 1,000 psi. These stress levels are below any of the strengths determined from the laboratory testing program (390 psi for tensile splitting tests and 3,660 psi for compression tests). The highest stresses under static loading are in areas of low strength as determined in the testing program. If for some reason the cross sectional area is significantly reduced, such as concrete deterioration as found in the history of Eisenhower Lock, the structural integrity would be in question. It is important for the SLSDC to maintain an aggressive maintenance program of replacing deteriorated concrete. In the near future, attention should be given to the repair of deteriorated concrete near the bottom of the lock walls at Eisenhower Lock.

For the seismic analysis, a geological-seismological investigation was conducted to find the strongest motions that the locks could expect to experience. There were no active faults either at the Eisenhower or Snell Lock sites or in the general area. The locks were determined to be in a high seismic area termed the Massena zone. The zone was given a floating earthquake of Modified Mercalli (MM) intensity of VIII which is the threshold of damage to properly built structures. Parameters for a Maximum Credible Earthquake (MCE) were assigned as horizontal peak motions of 0.83 g for acceleration, 35 cm/sec for velocity, and 12.4 sec for duration. The Operating Basis Earthquake (OBE) was assigned 0.46 g, 24 cm/sec, and 8.8 sec, respectively. A set of six accelerograms with response spectra was selected for use in the seismic analyses.

A response spectra seismic analysis was performed for a north chamber wall monolith at Eisenhower. For both the OBE and MCE conditions, catastrophic failure of the lock wall is unlikely to occur. No damage should occur during the OBE event with a maximum principal stress of 300 psi. There is a possibility of damage to the structure during the MCE event near the top of the structure where the sloping face

transitions into a vertical face. If damage does occur during the MCE event, no more than about 5 to 10 percent of the section would be damaged. Since, the MCE event has a low likelihood of occurrence and the damage level is low, it would be prudent to wait and make repairs to the chamber walls if such an earthquake event.

PREFACE

The Waterways Experiment Station (WES), US of Army Corps of Engineers, was authorized to conduct this study by the US Army Engineer District, Buffalo on 12 February 1990 by DA Form 2544, Number NCB-DA-90-16EJ.

Mr. Reed L. Mosher, Computer-Aided Engineering Division (CAED), Information Technology Laboratory (ITL), WES, was the Project Manager for the study. He was also the Principal Investigator for the static stress analysis and the primary author for this report. Messrs. Michael Pace and Kevin Abraham, both of CAED, helped in performing the static stress analysis. Mr. Tommy L. Bevins, Structural Analysis Group (SAG), Structural Mechanics Division (SMD), Structures Laboratory (SL) performed the seismic stress analysis under the direction of Dr. Robert L. Hall, Chief, SAG, SMD, SL. Mr. Bevins authored the section of this report describing the seismic analysis. Mr. Billy D. Neeley, Application Group (AG), Engineering Mechanics Branch, (EMB), and Dr. Robert H. Denson, formerly of Engineering Sciences Branch (ESB), Concrete Technology Division, SL, supervised the laboratory testing program. They were assisted by Messrs. Percy Collins, AG, Michael Lloyd, AG, Roy C. Gill, Cement and Pozzolan Group, ES, Sam Wong, Engineering Material Group (EMG), Ms. Judy Tom, EMG, and Ms. Linda Mayfield, Applied Mechanics Group, EM. Mr. Neeley authored the section of this report describing laboratory test results. Ms. Eileen M. Glynn, Rock Mechanics Branch (RMB), Soil and Rock Mechanics Division (SRMD), Geotechnical Laboratory (GL), logged the core and supervised drilling operation at Eisenhower and Snell Locks. Dr. E. L. Krinitzsky, Earthquake Engineering and Geosciences Division (EEGD), GL, conducted a geological-seismological investigation to find the strongest motions for the seismic analysis.

COL Larry B. Fulton, EN, was Commander and Director of WES during the preparation of this report. Dr. Robert W. Whalin was Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsuis degrees or Kelvins*
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F-32)$. To obtain Kelvin (K) readings, use; $K = (5/9)(F-32) + 273.15$.

STRUCTURAL EVALUATION OF EISENHOWER AND SNELL LOCKS, SAINT LAWRENCE SEAWAY, MASSENA, NEW YORK

PART I: INTRODUCTION

1. The Eisenhower and Snell Locks were constructed between 1955 and 1958 as part of an international cooperative effort to build the Saint Lawrence Seaway. The US portion of the project was authorized by the Wiley-Dondero Act of 13 May 1954. This act also created the Saint Lawrence Seaway Development Corporation (SLSDC) to construct, operate, and maintain the locks. SLSDC contracted with the Corps of Engineers (CE) to design and construct these locks. The CE has had a long history of providing engineering and review assistance to the SLSDC for the two locks. The SLSDC requested the CE through an Intergovernmental Agency Agreement (Appendix A.) to preform a structural evaluation study of the two locks and prepare a report detailing the findings of the study. The US Army Engineer Buffalo District (NCB) has overall responsibility for the study's coordination and project managing.

Purpose and Scope

2. The purpose of the structural evaluation of Eisenhower and Snell Locks is to determine the adequacy of the existing locks considering present condition and future needs, and to determine the advisability of their rehabilitation. The scope of this study focuses on the internal structural integrity of the chamber wall monoliths at each lock. The study reviewed the history of the Saint Lawrence Seaway Development Corporation's (SLSDC) structural repairs to these monoliths and evaluated the monoliths' current internal condition based on field investigations and detailed finite element analyses for both static and seismic conditions. Previous reports, including the Gannet Fleming reports, were reviewed to identify pertinent information and acceptable existing data for use in the current evaluation.

3. The study is divided into three phases:

a. Phase I - Field Investigation and Laboratory Testing Program: This phase of the study explored the current state of the in-situ concrete in the chamber wall monoliths and determined the concrete properties needed for the stress analyses in the subsequent phases of the study.

b. Phase II - Static Stress Analysis: In this phase of the study, the state of stress within the chamber wall monoliths is examined by performing nonlinear two-dimensional soil-structure interaction finite element analyses of two monoliths at Eisenhower Lock and one monolith at Snell Lock.

c. Phase II - Seismic Stress Analysis: Dynamic finite element analyses of two monoliths are performed to assess their structural integrity during an earthquake. To support the dynamic analysis, a geological-seismological investigation was conducted to provide the ground motions, time histories, and corresponding response spectra that could be expected during a seismic event in the region.

Detail Scope of Work

4. In this section, the general scope of work presented in the Intergovernmental Agency Agreement is expanded.

Phase I - Field investigation and laboratory testing program

5. The field investigation and laboratory testing program were conducted to provide the necessary parametric data to support the static and seismic analyses. The Mobile District conducted the drilling operation. Continuous six inch diameter cores were obtained from vertical core borings made at six representative locations. Four core borings were made at Eisenhower Lock (in monoliths - N53, N57, S19, and S17) and two at Snell Lock (in monoliths - N56 and S15). Each boring, except N56 at Snell, was approximately 125 ft deep and penetrated 10 to 15 ft into the bedrock beneath the locks to check for any unknown rock cavities. Boring N56 extended only 74 ft from the top of the wall. A total of approximately 750 lin ft of 6-in. diam core were obtained. Core drilling was performed from the top of the lock walls. For monoliths N53 and N57 at Eisenhower Lock and N56 at Snell Lock, the borings were located approximately 4 ft from the face of the lock wall to avoid the cable gallery. All borings were located to ensure no interference with the main culvert or any ports for

filling and emptying. All coring was accomplished with a single drill rig and with only one mobilization. Mobile District provided a borehole camera that was used to visually inspect the existing conditions of the concrete and rock core holes. WES had a field representative on site to log and inspect the cores. All cores were wrapped with material to resist drying. The cores were stored on site in space provide by SLSDC.

6. Two additional core borings were made at Eisenhower Lock in monoliths S19 and S17. These borings were located as close as possible to the back of the walls and were extended to a depth of approximately 20 ft. These borings were used to investigate the possibility of the existence of a crack near the top of the wall where the back of the walls change slope.

7. Once the coring was complete, samples were selected and shipped to WES for laboratory testing. The laboratory testing program consisted of tests to find unconfined compressive strength, Poisson's ratio, elastic modulus, splitting tensile strength, specific gravity, absorption, voids, and ultrasonic pulse velocity along with direct shear tests and petrographical examination for the concrete. For rock the testing program consisted of tests to find unconfined compressive strength, Poisson's ratio, elastic modulus, and ultrasonic pulse velocity along with several direct shear tests at the interface between the concrete and rock.

Phase II - Static stress analysis

8. The static stress analysis phase focused on the evaluation of internal structural integrity of the chamber wall monoliths at Eisenhower and Snell Locks. Of primary concern is the state of stress of the concrete in the chamber walls. The stress conditions in the chamber walls result from gravitational forces on the walls and from external forces applied by the backfill and water. The stress analyses were performed by using the finite element method. Three typical concrete chamber wall monoliths were analyzed, a north and a south wall at Eisenhower and a north wall at Snell. Each monolith was analyzed for three loading conditions: normal operation with upper pool in the chamber; normal operation with lower pool in the chamber; and the maintenance condition with the chamber fully dewatered.

9. The analyses were performed using the computer program *SOILSTRUCT* written by Dr. G. Wayne Clough and co-workers at Virginia Polytechnic Institute and

State University. *SOILSTRUCT* was specifically developed to model complex soil-structure interaction problems by the finite element method. The finite element computer program has been used to analyze a wide variety of problems, such as navigation locks, retaining walls, supported excavations, dams, tunnels, foundations, and cofferdams. The analyses were incremental soil-structure interaction analyses where the placement of the concrete and the backfill is simulated so as to obtain the induced stresses in the system. The incremental analysis applies the loads to the model in small increments to allow the nonlinear constitutive model for the soil and interface to adjust during the loading increment.

10. The study commenced by reviewing construction records and previous studies of the locks. From this information, the characterization of the backfill and foundation and a preliminary characterization of the concrete was made. The chamber wall monoliths were reviewed for selection of the monoliths for analysis. After selection of the monoliths, a finite element mesh was developed for the north wall at Eisenhower Lock. Once the mesh and modeling techniques had been proven to be sufficiently accurate, meshes for the other selected monoliths were generated. After the field investigation and laboratory testing had been completed, the final material characterizations were made for the analyses and each was reanalyzed for the three loading conditions. The results from the three analyses were processed and evaluated considering future rehabilitation of the walls and results are presented later in this section of the report.

Phase III - Seismic stress analysis

11. A response spectra seismic analysis was planned for either Eisenhower or Snell lock depending upon which structure was subjected to the highest seismic accelerations. If the earthquake ground accelerations were approximately the same, the seismic analysis would be conducted on Eisenhower Lock, since it has the poorest concrete conditions. To determine the strongest ground motions, a geological-seismological investigation was conducted to provide the peak values of acceleration, velocity, and duration for comparison of earthquake ground shaking between both sites. Time histories (accelerograms) and corresponding response spectra were

provided to represent site-specific cyclic shaking as it would be felt at the structure experiencing the strongest disturbance.

12. Once the most critical structure was determined, it was analyzed considering three loading conditions together with an operating basis earthquake (OBE). The three load cases are: normal operation with upper pool in the chamber; normal operation with lower pool in the chamber; and the maintenance condition with the chamber fully dewatered. Two different typical lock-wall monoliths were analyzed using two-dimensional (2-D) finite element procedures with a response spectra from the OBE. These analyses considered the 2-D dynamic behavior of the structure, hydrodynamic loading from the water in the lock, and the dynamic lateral earth pressures. The active dynamic soil pressures were computed using the Mononobe-Oakbe equation, and the passive pressures were computed from either a Coulomb or log spiral analysis. The foundation of the lock is considered rigid. Static stresses from Phase II were used with the dynamic stresses in order to determine the total state of stress within the structures.

13. These analyses provided a preliminary structural evaluation of this structure for a seismic event at the site. If the total state of stress is within the allowable limits of the material, no further seismic stress analyses will be necessary. However, if these analyses indicate that cracking is likely, a seismic analysis using a time-history acceleration record and a nonlinear concrete model should be considered. Such an analysis is beyond the scope of work for this study and would require additional funds.

Organization of the Report

14. In Part II, the background and historical information on Eisenhower and Snell Locks is presented. Part III summarizes the results of the Field Investigation and Laboratory Testing Program phase of the work. Part IV describes the procedures for the the static stress analyses and presents the results of these analyses. Part V describes the procedures for the dynamic stress analyses and reports the findings. Part VI summarizes the findings and presents the conclusions drawn from the study along with recommendations on need for future rehabilitation of the locks.

PART II: BACKGROUND

Introduction

15. The Eisenhower and Snell Locks were constructed between 1955 and 1958 as part of an international cooperative effort to build the Saint Lawrence Seaway. The US portion of the project was authorized by the Wiley-Dondero Act of 13 May 1954. This act also created the Saint Lawrence Seaway Development Corporation (SLSDC) to construct, operate, and maintain the locks. SLSDC contracted with the Corps of Engineers to design and construct these two locks.

16. The Eisenhower and Snell Locks are located in the Wiley-Dondero Canal portion of the Saint Lawrence River just north of Massena, NY, (Figure 1). The locks are about 4 miles apart and together they allow vessels to transit around the Saint Lawrence Power Project. Figure 2 illustrates the locations of the locks, and the Long Sault Dam and Moses-Saunders Power Dam that are the hydropower portions of the Saint Lawrence Project.

17. The Eisenhower and Snell Locks have lifts of 38 to 42 ft and 45 to 49 ft, respectively. The chamber dimensions are 80 ft in width, 860 ft in length from upstream miter gate to downstream miter gate, and the locks have 30 ft of water depth over the sills.

18. The locks are concrete gravity type walls founded on rock. Both locks have mitering type gates, fender booms, and stop log closures. Eisenhower Lock has a vertical lift gate to prevent loss of Lake Saint Lawrence in the event something should happen to the miter gates. There is also a vehicular tunnel built through the upstream miter gate sill at Eisenhower Lock. The filling system consists of upper sill intakes, tainter valves, wall culverts, and side ports. This system empties into a floor diffuser system below the lower miter gate. The guide walls are of the concrete gravity type except the downstream guide wall at Snell Lock that is a concrete cap set on steel sheet piles.

Historical Problems and Repairs

19. Eisenhower Lock has experienced problems both with surface concrete deterioration and with structural cracks from the filling/emptying culvert to the land side of both lock walls. Snell Lock has not had a surface concrete deterioration problem, but has experienced the same problem with culvert cracks. This section will briefly present the surface concrete deterioration problem at Eisenhower Lock, and the structural crack problem at both the Snell and Eisenhower Locks.

Concrete deterioration

20. The concrete deterioration problem at Eisenhower Lock has been linked to the natural cement used in the concrete mix. The mix at Eisenhower contained 25 percent by weight natural cement. After review of the available data and reports on the concrete deterioration, the mechanism of the concrete deterioration is freezing of water in the pores of the concrete. While the mechanism of the concrete deterioration is clear, the reason the concrete at Eisenhower lock is less resistant to deterioration than the concrete at Snell Lock is less clear.

21. The concrete mix both Eisenhower and Snell Locks varies throughout the structures depending on the location. The concrete mix design was the same at both locks except for the 25 percent by weight of natural cement. The concrete contain 4 to 6 percent entrained air. A detailed investigation of concrete at the two locks was conduced by the US Army Engineer Waterways Experiment Station (WES) (Buck, Mather, Thorton 1967).

22. Both the Corps (Buck, Mather, Thorton 1967) and Harza Engineering Company (Harza 1981) sighted the slow development of the strength of the concrete at Eisenhower lock as the most plausible reason for the lower resistance to frost damage. The available evidence from the construction records and laboratory experiments show that the Eisenhower concrete developed strength more slowly than did the Snell concrete. Based on the construction data, it would take about 12 and 37 days, respectively, for the Snell and Eisenhower exterior grade concrete made in 1956 to reach a strength of 3,000 psi (Buck, Mather, Thorton 1967). Buck, Mather and Thorton reported that although laboratory tests had not shown one strength level

to be required for adequate durability of concrete for all mixtures, the data did show that a considerable strength development was necessary before concrete becomes durable under freeze-thaw conditions. The value of 3,000 psi compressive strength was used to illustrate the different rates at which equal levels of strength maturity would be attained between the concrete in Eisenhower and Snell Locks. The 3,000 psi compressive strength value had been found at the time of the report to produce durable concrete in the laboratory freezing and thawing tests. However, it stated that under the severe exposure conditions actually experienced at Eisenhower and Snell Locks, a greater maturity would have probably been required.

23. The concrete was required to be kept at a temperature above 40 °F for 5 days and above freezing for 14 days. Buck, Mather, and Thorton reported that climatological data at Eisenhower and Snell Locks show that the cores for the concrete placed 24-27 September and 22-26 October 1956 would have been subject to freezing at an age between 14 and 18 days. The exterior concrete at Eisenhower Lock placed during 1956 had an average 28-day compressive strength of 2,812 psi as compared at 28-day compressive strength at Snell Lock of 3,954 psi. Yet by 1966, samples of nondeteriorated concrete from comparable locations within Eisenhower and Snell Locks had compressive strength approaching one another, 5,160 psi (range 4,190 to 5,860 psi) and 5,550 psi (range 4,760 to 6,450 psi), respectively (Buck, Mather, Thorton 1967).

24. This was regarded by Buck et al (Buck, Mather, Thorton 1967) as the most probable reason for the low durable of the concrete at Eisenhower Lock. If the concrete had matured enough, it should have been just as frost resistant as the Snell concrete has proven to be in service. The freezing of the low frost resistant concrete had the effect of introducing additional void space, such as micro cracks, that would not otherwise be present. This additional void space, beyond that the entrained air-void system had been provided to protect against, would provide the location of additional freezable water that could freeze and produce progressive deterioration of the concrete.

25. Harza Engineering sighted the slow development of strength in the concrete at Eisenhower Lock as the most plausible cause for the low resistant to frost

action and resulting low durability of the concrete. Based on their calculation of maturity factor as defined by ACI Committee 306, March 1978, it appeared to them that concrete placed after 1 October and before 15 April 1957 has less than desirable resistance to damage by frost.

26. At the time the locks were constructed, the winter curing and protection techniques employed were common engineering practice. The contractor was required to maintain concrete temperatures above 40 °F for 5 days followed by additional curing above 32 °F for 9 days.

27. Extensive concrete repairs have been made to the chamber faces, filling and emptying culverts, gate recesses, pintle bases, and sills at Eisenhower Lock. SLSDC has had aggressive program to repair and replace deteriorated concrete. The concrete deterioration at Eisenhower Lock will be a continuing problem.

Culvert cracks

28. In January 1967, during inspections of the Eisenhower Lock filling and emptying culverts by SLSDC and CE personnel immediately after winter dewatering, a continuous crack was observed along the landward-ceiling corner of the culvert in the north wall. Further investigation revealed that these cracks were continuous from the culvert through to the exterior backfilled face of the lock wall. At the time, the crack leaked water in varying amounts along its entire length and fresh spalls of concrete were found lying on the culvert floor beneath it. Subsequent detailed inspections and other pertinent investigations revealed that the crack extended along the culvert between the upper and lower valve monoliths.

29. After initial discovery of the crack in the north culvert, a close inspection was made in the south culvert. The same kind of crack as existed in the north culvert was present in the south culvert, at its landward-ceiling corner. Its longitudinal extent were the same as that of the north culvert crack. Examination of the Snell Lock culverts revealed similar cracks in that lock.

30. With these cracks extending through to the backfill, the overall stability of the lock walls became a matter of serious concern. Under certain conditions, all wall loads must be taken on the 15-ft-thick section between the culverts and the faces of the lock chambers. This was thought to be especially serious with respect to

Eisenhower Lock where portions of this section were deteriorated and thus less capable of supporting the imposed loads. The core boring program underway concerning the problem of deterioration was enlarged to include exploration of these cracks. To obtain additional data on the extent of the cracks and condition of the surrounding concrete, joint meters were installed across the cracks to measure changes in the size of the cracks during lock operations, bar joints were installed across the lock chambers to measure relative movements of the lock walls and an inclinometer was used to measure tilting of the lock walls during operation, alignment control was set up whereby any lateral displacement of the wall could be measured, piezometers were installed in the backfill areas to provide information on saturation levels and drainage patterns, and correlated flow measurements were taken of flows in the backfill drains.

31. Because of this information in combination with the knowledge gained by the concrete survey in 1966-67, a determination was made by an SLSDC convened Board of Consultants that a complete rehabilitation program was necessary to guarantee continued structural integrity and stability and to assure ability to operate the locks. In a letter of June 26, 1967 from the Administrator, the CE was requested to perform the necessary design and contracting services concerning the proposed rehabilitation program for the Eisenhower and Snell Locks to restore the locks to a condition of full stability.

32. Priority was given to Eisenhower Lock. The rehabilitation work for the crack consisted of placing post-tensioned anchors across the culvert crack, . This was accomplished during the winter shutdown of 1967-68 by contract with Peter Kiewit & Sons. Similar post-tensioned anchors were placed across the culvert cracks at Snell Lock during the winter of 1968-69 under contract with Morrison-Knudsen.

Post-tensioned anchors - lock wall monoliths

33. In the winters of 1967-68 and 1968-69, post-tensioned anchors were installed in Eisenhower and Snell Locks, respectively. The north and south walls of the Eisenhower and Snell Locks have 14 monoliths with narrow tops and sloping backs (Figure 3). These walls in the chamber portion of the locks are 606 ft long. The typical chamber monoliths with anchors are outlined on a plan view of Eisenhower Lock in Figure 4. Eighty-two and eighty-three anchors were installed in the north and

south walls, respectively, of each lock. Six 636 kips anchors were installed in each monolith. The average spacing of the anchors is 7.33 ft.

34. Review of data and stability analyses show that the saturation level in the backfill was at el* 221.0 ft at the time of anchor installation. This elevation is 16 ft higher than was designed for originally. Recent field inspection of drainage pipe and dye tracing study by Gannet Fleming Geotechnical Engineering, Inc.(1986) of the drainage blanket show that they are operational and continuous. From historical data and recent observations it was determined that the static groundwater level is at the drain invert in the drainage blanket for the soil below the blanket. This data also shows that the soil is saturated up to 18 ft above the drain in the same locations. These high piezometer levels observed in the upper portion are the result of a perched water table fed by the water level in the nature soil. While the drainage blanket and pipe are functioning, they are not connected to the soil above the drainage blanket.

35. In February 1989, the SLSDC and CE conducted an anchor investigation program at Eisenhower Lock. The objective of the investigation was to determine whether or not the post-tensioned anchors in the chamber monoliths at Eisenhower Lock have sustained any significant corrosion due to water leakage through the existing culvert cracks. Of the 165 anchors in Eisenhower Lock, two anchors were examined, one in monolith N-51 and one in monoliths S-17, at locations near the greatest amount of leakage through the existing culvert cracks. Significant corrosion was considered to have the greatest potential at these locations. The investigation consisted of excavating the concrete from inside the culvert to expose a short section of each anchor for visual inspection and dimensional measurements.

36. Results of the anchor investigation showed that the grout was intact and completely surrounded the anchor strands in the exposed areas. The anchor strands were observed to be as shiny as new and there was no evidence of any surface corrosion or pitting. Based on the results of this investigation, it was concluded that the anchors are in excellent condition. It was further concluded that post-tensioned

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* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

anchors in Eisenhower Lock should remain structurally sound and should adequately serve the anticipated life expectancy of the lock. In any future structural evaluation of the lock, the existing anchors should be assumed 100 percent effective.

PART III: FIELD INVESTIGATION AND LABORATORY TESTING PROGRAM

Site Visit

37. In February 1990, representatives from WES, Buffalo District (NCB), North Central Division (NCD), Headquarters (HQ), and the Saint Lawrence Seaway Development Corporation (SLSDC) inspected the Eisenhower Lock while it was dewatered for winter maintenance. The lock was dewatered for repair to the concrete surrounding the miter gates and to replace surface concrete at the toe of the same chamber monoliths. Poor concrete was visible in some locations. A number of locations of unsound concrete were found during the inspection. The recently repaired surface concrete appeared to be sound.

Drilling Operation

38. After completion of inspection of the locks, boring locations were assigned in areas that were representative of the concrete placed after 1 October 1956 and before 1 April 1957. Four boring locations for Eisenhower Lock are shown in Figure 5 and two boring locations for Snell Lock are shown in Figure 6. Personnel from the US Army Engineer District, Mobile, drilled the borings using diamond-tip core barrels. Approximately 750 lin ft of 6-in.-diam cores were obtained from vertical borings. While, larger diameter cores would have been preferable because of a maximum aggregate size of 6 in., but their cost was prohibitive. The borings were located so as not to interfere with the main conduit or any ports for filling and emptying. Personnel from the WES logged the cores on ENG Form 1836. The boring logs are presented in Appendix B. Pertinent information describing each boring including boring number and depth is presented in Table 1.

39. The cores were preserved and handled using procedures described in Test Standard RTH 103-80 (WES 1980). After removal from the core barrel, each core was marked to indicate boring location and depth. Loss of moisture was prevented by

wrapping the core in thin polyethylene, followed by cheese cloth, and then coating with a lukewarm wax. The cores were then placed in a wooden box, cushioned with vermiculite, and shipped to the WES for testing.

40. Two core borings were made at Eisenhower Lock in monoliths S19 and S17 to investigate the possibility of the existence of a crack near the top of the wall where the back of the walls change slope. The borings were located as close as possible to the back of the walls and were extended to a depth of approximately 20 ft. No cracks were found.

41. A borehole camera that was used to visually inspect the existing conditions of the concrete and rock core holes. An examination of the video tapes of the borehole camera inspection showed no voids or unusual conditions in the concrete.

Test Specimens and Laboratory Testing

42. A visual examination of all cores was made in the laboratory to supplement the field boring logs and to assist in the selection of representative test specimens. Concrete test specimens were selected based upon the physical condition of the concrete and depth in order to obtain representative properties throughout the structure. Figure 7 is a schematic showing the general location of samples selected for testing. Rock specimens were selected to be representative of the bedrock in close proximity to the base of the structure. Forty-eight concrete specimens, six rock specimens, and two concrete-bonded-to-rock specimens were tested from Eisenhower Lock. Twenty-two concrete specimens and two rock specimens were tested from Snell Lock.

43. Each specimen was visually examined prior to any destructive testing. The results of the visual examination are given in Table 2. The laboratory testing consisted of the following conducted in accordance with the appropriate test method:

Property	Test Method Rock	Concrete
Unconfined compressive strength	RTM 111	CRC-C 14 (ASTM C 39)
Elastic modulus and Poisson's ratio	RTM 201	CRD-C 19 (ASTM C 469)
Splitting tensile strength	---	CRD-C 77 (ASTM C 496)
Specific gravity, absorption, voids	---	CRD-C 23 (ASTM C 642)
Ultrasonic pulse velocity	RTM 110	CRD-C 51 (ASTM C 597)
Direct shear	RTM 203	CRD-C 90

Direct shear testing was performed on intact concrete specimens and concrete-bonded-to-rock specimens. Results of the laboratory testing are given in Tables 3 to 8.

General Comments for Concrete

Eisenhower Lock

44. There appears to be considerable variation in the quality of concrete in both the north and south sides of the lock. Compressive strengths range from above 6,000 psi to below 4,000 psi, while splitting tensile strengths range from above 700 psi to below 400 psi. The top 15 ft of concrete appears to be dense and well consolidated. At greater depths, the concrete exhibits varying degrees of voids and poor aggregate-paste interface. Some of the larger voids could have resulted from poor consolidation. However, the mortar fraction of the concrete in these areas appears to be porous, grainy, and generally of poor quality. The test data indicates the poorest quality concrete to be in the lower part of the lock, primarily in areas 15 ft above the bedrock down to the bedrock. Variable quality is also indicated by ultrasonic pulse velocity and voids content. An increase or decrease in compressive strength in a particular area was not always confirmed by an increase or decrease in splitting tensile strength, ultrasonic pulse velocity, or voids content. Average physical properties of the concrete in the upper 15 ft as compared to the lower 15 ft as well as the overall average are given below:

Test	Upper	Lower	Percent Difference	Overall Average
Unit weight, pcf	159.2	155.8	-2.1	157.8
Pulse velocity, fps	18,210	16,834	-7.6	17,505
Compressive strength, psi	6,050	4,070	-32.7	5,230
Splitting tensile str., psi	620	570	-8.1	580
Elastic modulus, x 10 psi	7.55	4.65	-38.4	5.70
Poisson's ratio	0.28	0.22	-21.4	0.23
Percent voids	9.1	10.4	-4.3	10.3

45. There was no bond at the concrete-bedrock interface in three of the four borings. The bedrock was primarily a hard, dense, dark gray dolomite with occasional layers of weak clay or shale.

46. There was little difference in the physical properties on the north side of the lock as compared to those on the south side. The average compressive strength of the concrete in the south side was approximately 10 percent higher than that on the north side. Other properties differed by approximately 4 percent or less. Average properties for the concrete on the north and south sides are given below:

Test	North side	South side
Unit weight, pcf	158.3	157.4
Pulse velocity, fps	17,790	17,248
Compressive strength, psi	4,930	5,500
Splitting tensile strength, psi	585	580
Elastic modulus, x 10 psi	5.75	5.65
Possion's ratio	0.22	0.23
Percent voids	10.3	10.3

Snell Lock

47. The general quality of concrete varies with depth on both sides of the lock, but appears to be somewhat better than that of the Eisenhower Lock. Compressive strengths range from above 8,000 psi to below 4,000 psi, while splitting tensile strengths range from above 900 psi to below 500 psi. The top 15 ft of concrete appears to be dense and well consolidated on the north side of the lock. At greater depths, the concrete exhibits some minor voids and varying strengths. On the south side of the lock, varying degrees of voids, poor aggregate-paste interface, and strengths are indicated from the top surface down to approximately 5 ft above the bedrock. The concrete just above the bedrock is of a poorer quality. There was no bond at the concrete-bedrock interface. There appears to be no correlation between compressive strength, splitting tensile strength, ultrasonic pulse velocity, nor voids content. An increase or decrease in compressive strength in a particular area was not always confirmed by an increase or decrease in splitting tensile strength, ultrasonic pulse velocity, or voids content. Average physical properties of the concrete are given below:

Test	Overall Average	Lower	Percent Difference
Unit weight, pcf	158.3	158.3	0
Pulse velocity, fps	17,810	17,046	-4.3
Compressive strength, psi	6,620	3,730	-43.7
Splitting tensile strength, psi	650	495	-23.8
Elastic modulus, x 10 psi	6.85	4.60	-32.8
Poisson's ratio	0.25	0.20	-20.0
Percent voids	11.0	11.6	5.5

48. Concrete on the south side of the lock appears to be inferior to that in the north side. Splitting tensile strength, elastic modulus, Possion's ratio, and voids content differed by more than 10 percent while unit weight, pulse velocity, and compressive differed by 4 percent or less. Average concrete properties on the north side and on the south side of the lock are given below:

Test	North side	South side
Unit weight, pcf	158.7	157.9
Pulse velocity, fps	18,127	17,494
Compressive strength, psi	6,750	6,480
Splitting tensile strength, psi	710	590
Elastic modulus, x 10 psi	7.20	6.45
Possion's ratio	0.26	0.23
Percent voids	10.3	11.7

Petrographic Examination

49. Cores from both Eisenhower and Snell Locks were taken for evaluation and examination. Portions of two cores from Eisenhower and one from Snell were

selected for examination. The complete test results are presented in Appendix G. The concrete was air-entrained. The air content for the samples examined ranged from 2.3 to 2.8 percent. The low air content of the concrete is not believed to be of any significance. Previous average air contents reported by Buck, Mather, and Thorton (Buck, Mather, Thorton 1967) were 3.5 percent for the two lock structures. It may be due to the relatively small area of sampling. The examination show that the concrete from both structures are similar and free from damaging cement-aggregate reactions and other deleterious reaction. The concrete from the two structures are similar.

Summary

50. The concrete testing program shows that the three principal properties needed for the finite element analyses, compressive strength, tensile strength, and modulus of elasticity, Figures 8, 9, and 10, respectively, decrease with depth in both Eisenhower and Snell Locks. The average compressive strength for Eisenhower Lock is less than that of Snell by 31 percent, 5,230 psi versus 6,620 psi. The lower value at Eisenhower is biased by the larger number of test specimens from the lower portion of the wall when compared with specimen from Snell, six from Eisenhower and only one from Snell. Using the average compressive strength from specimens taken from the upper portion of Eisenhower borings and comparing that to the Snell average, the difference is only 9 percent which is approximately the same percent difference reported by Buck, Mather, and Thorton (Buck, Mather, Thorton 1967). This same trend of bias by the larger number of specimen from the lower portion of the borings from Eisenhower is found in the overall average value for the modulus of elasticity. This trend is not seen in the tensile strengths. The average tensile strengths from the lower to upper portion of the borings from Eisenhower only vary by 8 percent. The test values for the modulus of elasticity and the unit weight may have been influenced due to the aggregate size when compared to the diameter of the cores which may result in higher test values.

PART IV: STATIC STRESS ANALYSIS

Introduction

51. This section presents the results of the static, nonlinear, soil-structure interaction analysis of three typical concrete chamber wall monoliths, a north and a south wall at Eisenhower Lock and a north wall at Snell Lock. The static stress analysis of Eisenhower and Snell Locks focuses on the evaluation of the internal structural integrity of the chamber walls monoliths. The objective of the study is to determine the current state of stress of the concrete in the chamber monoliths at Eisenhower and Snell Locks using the finite element method and to check these stresses against the strength of the concrete.

Problem Description

52. The Eisenhower and Snell Locks are located in the Wiley-Dondero Canal portion of the Saint Lawrence River just north of Massena, NY. The locks are about 4 miles apart with Snell Lock located downstream of Eisenhower Lock (Figure 2). Eisenhower and Snell Locks are sister locks with chamber dimension of 80 ft in width and 860 ft in length. The lock walls are approximately 110 ft high and 63 ft in width at the base. Figures 11 and 12 show typical cross sections through the lock chamber at Eisenhower and Snell Locks, respectively. The chamber walls are unreinforced mass concrete sections with large filling and emptying culverts, smaller drainage and supply culverts, and a cable gallery at the top of the north walls. The concrete monoliths are founded on dolomite rock which forms the chamber floor of the locks. The backfill behind the lock walls is a compacted glacial till. The natural soil at the site is a glacial till. Figures 13, 14, 15, and 16 show typical chamber wall sections for the two locks. Eisenhower Lock has a high pool level at el 240 ft and a low pool level at el 200 ft. Snell Locks has a high pool level at el 200 ft and a low pool level at el 155 ft.

53. As discussed in Part II, Eisenhower and Snell Locks have suffered structural cracking of the concrete. The chamber wall monoliths have a continuous crack

along the landward ceiling corner of the filling and emptying culverts. The crack extends from the culvert to the surface of the backfilled face of the monolith. The primary loading on the chamber walls that could cause the formation of the crack is from the backfill and the groundwater in the backfill. The backfill is a compacted glacial till with unusually high unit weight, 145 to 150 pcf. In situ tests by Gannett Fleming Engineering, Inc. (1986) to determine the horizontal stresses, i.e., lateral earth pressure, in the backfill showed that the horizontal stresses may be 1.5 to 2.0 times the effective vertical overburden pressure. This suggests a high degree of compaction of the backfill. An additional factor is that the groundwater level in the backfill is approximately 20 ft higher than anticipated in the original design.

54. With the backfill as the primary source of loading, the assessment of the stress in the concrete of the chamber walls involves significant soil-structure interaction. Because of the nonlinear stress-strain behavior of the soil and the formation of the crack in the concrete, a nonlinear soil-structure interaction finite element analysis is necessary to accurately estimate the stress in the chamber wall monoliths. Conventional equilibrium method and linear elastic finite element models cannot capture these important aspects of this problem.

55. The analyses for the assessment of static stresses in the chamber wall monoliths were performed using the computer program SOILSTRUCT. SOILSTRUCT is a general purpose finite element computer program for two-dimensional, plane strain analysis of soil-structure interaction and soil-inclusion interaction problems. It calculates displacements and stresses due to incremental construction and/or load application and is capable of modeling nonlinear stress-strain material behavior. The simulation of incremental construction may include embankment construction or backfilling, the placement of layer(s) of a reinforcement material during backfilling or embankment construction, dewatering, excavation, installation of a strut or tie-back anchor excavation support system, removal of the same system and the placement of concrete or other construction materials. The incremental loading simulation may consist of the application of concentrated loads, boundary pressures or loads due to temperature changes in nonsoil materials.

56. The initial version of *SOILSTRUCT* was developed by Professors G. W. Clough and J. M. Duncan for use in the analysis of Port Allen and Old River U-frame locks (Clough and Duncan 1969). This version of the program reflects modifications made in conjunction with a number of projects at WES to expand the capabilities of the finite elements constitutive models, load vector formulation algorithms, the size of problem which may be analyzed, and the transfer of input, output, restart and plot data files by means of disc storage. A detailed description of the program is provided in Appendix C.

Finite Element Model

57. The study commenced by reviewing construction drawing and records and the reports from previous studies of the locks. From this information, the characterization of the backfill and foundation and a preliminary characterization of the concrete was made. The next step in the analysis was to select the dimensions and elevations for the chamber wall monoliths to be analyzed. NCB requested the analysis of three chamber monoliths, a north and south at Eisenhower Lock and one from Snell Lock. In the detail scope of work, a south monolith at Snell Lock was scheduled for analysis. A review of the data revealed that the north chamber monoliths have a slightly higher backfill behind them resulting in the decision to analyze a north monolith at Snell Lock. Figures 17, 18, and 19 show the basic elevations and dimension for each lock wall sections selected for analysis. The dimensions and elevations are not for specific monoliths at Eisenhower and Snell Locks, but are lower bound possibilities formed from the typical sections shown in Figures 13 through 16. The boring data shows the rock surface to range from el 139 to el 143 ft at Eisenhower Lock and from el 99 to el 102 ft for Snell Lock. The Eisenhower models used el 140 ft for the rock surface and Snell model used el 100 ft for the rock surface. Figures 20, 21, and 22 shows the overall model configuration including the wall, backfill, and foundation. The backfill surface begins at the top of the wall for both locks and connects to the surface of the natural soils. The surface of the natural soils and the slopes of the excavation (Figures 20, 21, and 22) are consistent with preconstruction drawings for the two locks.

58. This information provided the basic data to construct the finite element meshes for three monoliths to be analyzed. Symmetry about the center line of the chamber allows for modeling of only one wall of the lock at a time. This significantly reduces the required mesh size. Each mesh contains a section through a lock wall and a representative portion of the foundation rock, backfill, and in situ soil. One of the side boundaries of the meshes is at the center line of the lock chamber. The other side boundary is 500 ft from the center line of the lock, in a position such that simulation of the construction of the lock wall and backfill placement should not effect the stresses near this boundary. The rock foundation extends 200 ft below the base of the lock walls.

59. Construction of the mesh for the north chamber wall monolith at Eisenhower Lock, the first mesh generated, consisted of several stages with certain constraints applied to the configuration of the mesh. Initially, a mesh was constructed with approximately half the elements of the final mesh and a backfill placement was simulated to test for accuracy. The mesh had to allow for the simulation of a crack extending from the landward ceiling corner of the filling and emptying culverts to the surface of the backfilled face of the wall. Special interface elements had to be used between the wall and the foundation and between the wall and the backfill.

60. Clough and Duncan (1969) found that an accurate mesh for the simulation of backfill placement was obtained if the stresses in the soil elements in the mesh did not vary more than 10 percent during a placement of a lift. Maintaining this criteria allows the soil constitutive model in the finite element computer program to follow the nonlinear stress-strain response of the soil. The initial mesh was refined until the changes in stresses were below 10 percent. Other criteria followed in the construction of the mesh was maintaining an aspect ratio less than 4 in the wall and surrounding soil. The QM5 element used in SOILSTRUCT is accurate for aspect ratios up to eight. The highest aspect ratios in the wall, less than four, are along the slope of backfilled face of the wall. In this area, the criteria was to have four or more elements parallel to the crack to allow for accurate prediction of stresses in the direction of the crack.

61. Figure 23 shows the mesh for a section through a typical chamber monolith of the north lock wall at Eisenhower Lock. The mesh consists of 2,505 nodal points to define 2,200 two-dimensional elements representing the lock wall, soil, rock, and 221 one-dimensional interface elements. This was the first mesh generated and was used to set a minimum standard for the two remaining meshes. Figure 24 shows the mesh for a section through a chamber monolith of the south lock wall at Eisenhower Lock. The mesh consists of 2,898 nodal points to define 2,818 two-dimensional elements representing the lock wall, soil, rock, and 290 one-dimensional interface elements. Figure 25 shows the mesh for a section through a chamber monolith of the north lock wall at Snell Lock. The mesh consists of 2,353 nodal points to define 2,278 two-dimensional elements representing the lock wall, soil, rock, and 245 one-dimensional interface elements. The interface elements are at the interface between the concrete and soil backfill, concrete and rock foundation, and soil and bedrock.

Simulation of the Construction and Loading Conditions

62. The finite element modeling technique used for this study is a nonlinear analysis, where loads are applied incrementally to simulate the actual conditions in the field. The key stages in the modeling are the initial conditions after excavation for the construction of the lock, placement of the lock wall, placement of the backfill, raising of the water table in the backfill, formation of the crack, placement of the anchors, and operation of the lock. These are depicted schematically in Figure 26.

63. To establish the in situ stresses after the excavations for the construction of the locks, the natural soil was placed incrementally in the configuration of the excavations. This established the necessary in situ stresses to start the modeling of the structure and the backfill placement and is more expedient than trying to model the excavation process. During this stage of the analysis, the elements in the rest of the mesh are weightless with no stiffness. After establishment of the natural soil and excavation, the lock wall is placed under gravity load. At this point, all displacements in the mesh are set to zero, thus creating the reference point for the remaining stages of construction and loading simulation.

64. The placement of backfill is simulated by placing the soil in load steps or lifts of one row of elements each. Five iterations for each lift were used to permit the stress-strain curves of the soils to be followed as closely as possible. This number of load steps and iterations has been found to generate an accurate response for earth pressures (Clough and Duncan 1969). In order to develop sufficient earth pressures on the lock walls to cause a crack to develop, the effects of compaction had to be included in the analysis. The program did not have a procedure to account for compaction, thus a procedure had to be developed. Elements above the lift being placed are considered inactive and have zero weight and no stiffness.

65. Compaction equipment moving across a backfill adjacent to a wall induces additional lateral earth pressures on the wall. When the compaction equipment moves away from the wall, a portion of the additional lateral earth pressures continues to act on the wall due to the inelastic nature of the soil. A typical pressure distribution against a wall resulting from compaction is shown in Figure 27. This is similar in shape to the pressure distribution reported in the Gannett Fleming report (1986) from the pressuremeter and hydrofracture testing. A simplistic approach was taken to simulate the effects of compaction in the backfill for this study. The procedure used was to increase the vertical forces being applied during simulation of the placement of a lift of the backfill. This was accomplished by increasing the unit weight of the backfill lift being placed by a magnification factor. The magnitude of the factor was determined by performing a series of analyses where the factor was increased until the minor principal stress at the landward ceiling corner of filling and emptying culvert reaches a value that would initiate a crack.

66. After placement of the backfill, the water level in the backfill was raised. The increase in water level was simulated by applying the water pressure in the backfill resulting from the change in elevation of the water table and applying boundary pressures to the back and base of the lock wall to equal the hydrostatic head due to the change in water table. The static groundwater level in the backfill below the drainage blanket is at the drain pipe invert in the drainage blanket. For the soil above the drainage blanket, the location of the perched water table, the groundwater level is set equal to the groundwater level in the natural soil.

67. Once it was determined that the earth pressures were sufficient to initiate the crack, another complete analysis was performed with zero strength and stiffness in tension along the crack. This provided the state of stress in the lock wall after the crack had occurred. Next, the placement of the post-tensioned anchor was simulated by applying point loads to the lock wall equal to the value for post-tensioning of the anchors in the field. In the subsequent load step, the stiffness of the anchor is activated. Six 636-kip anchors were installed in each monolith. The average spacing of the anchors is 7.33 ft. *SOILSTRUCT* uses the plain strain assumption. Thus, the load applied to simulate the post-tensioning of the anchors is equal to the post-tensioning loads, 636 kips, divided by the average spacing of 7.33 ft. The force due to post-tensioning of anchors is applied at the head of the anchor, while an equal but opposite force is applied to four nodes below the crack in the direction of the line of action of the anchor.

68. Operation loading conditions evaluated are a result of raising the water level in the chamber of the lock from the dewatered condition to the lower pool water level and then to the upper pool level. These are simulated by applying boundary pressures to the chamber wall face, surfaces of the culverts, and base of the chamber wall monolith equal to the hydrostatic water pressure for the different pool levels.

Soil and Concrete Parameters for Finite Element Analyses

69. The finite element models require input parameters for the concrete in the lock walls, foundation rock, backfill, and natural soil at the site that describes their stress-strain characteristics, unit weights, and initial stress conditions. These parameters were determined from the laboratory tests described in Part III and from the reports provided by the SLSDC and NCB.

Unit weights

70. The in-place relative soil density of the backfill soil has been a point of controversy for sometime because of the high values measured by in-place tests. Assumed wet densities from previous studies have ranged from 125 pcf in the 1955 Corps Design Memorandum to 140 pcf used in Harza Engineers' study (1981).

Measured backfill density values from in-place tests range from a low of 135.5 pcf to a high of 150.6 pcf. Measured values from laboratory tests of reconstituted soil samples showed an average maximum dry density ranging from approximately 135 to 140 pcf that would result in wet density values ranging from approximately 144 to 149 pcf. Based on this information, a wet unit weight of 148 pcf was used in these analyses.

Angle of internal friction and cohesion

71. Because of the difficulty involved in obtaining undisturbed samples, laboratory testing has only been performed on reconstituted samples. Measured values for the angle of internal friction from reconstituted samples ranged from 35 deg reported in the Corps Post Construction Report (US Army Engineers, Buffalo District 1957) to 42 deg reported by Empire Soils Investigations, Inc. (1985) and from zero to 5.4 ksf for cohesion. During the engineering study by Gannett Fleming Geotechnical Engineers, Inc., (1986) a value of 45 deg for angle of internal friction and zero cohesion was used in their stability computations. This angle of internal friction value was estimated from the results of pressuremeter tests.

72. The report by Empire Soils Investigation, Inc. (1985) recommended an angle of internal friction of 40 deg and zero cohesion, based on the average of three sets of triaxial tests of reconstituted samples (37, 40, and 42 deg, and 0,0, and 0.3 ksf, respectively). Our reanalysis of these tests, Appendix D, shows the angle of internal friction ranging from a low of 35.2 deg with cohesion of .23 ksf to a high of 41.7 deg with a cohesion of 1.5 ksf. The greater strengths were for the materials classified as SM. The lower strength materials were classified as CL.

73. It has been found that the angle of internal friction obtained from a triaxial test is from 4 to 9 deg smaller than from a plane strain test. Plane strain conditions develop along the chamber lock wall in this study. Therefore, the values from triaxial tests will under estimate the angle of internal friction of the backfill behind the lock walls. A conservative estimate of the plane strain angle of internal friction may be found using the value from a triaxial test in the correlation:

$$\phi_{ps} = 1.5\phi_{tx} - 17$$

for an angle of internal friction greater than 34 deg (Lade 1976). Using this correlation the plane strain angle of internal friction from a triaxial test performed by Empire Soils Investigation, Inc. would be 38.5, 43, and 46 deg, respectively, and a similar range for our reanalysis. In light of this, the value of 45 deg for angle of internal friction used by Gannett Fleming Geotechnical Engineers, Inc. (1986) seems reasonable and was used in this study. The reanalysis of the Empire Soil tests showed significant values of cohesion. It was felt that a cohesion of 0.5 ksf should be used in the analyses.

Stress-strain parameters for the soil

74. Using the procedures outlined by Duncan et al. (Duncan 1970) the necessary stress-strain parameters can be computed from the results of drained triaxial tests. These procedures were used to determine the stress-strain parameters from the triaxial tests reported by Empire Soil Investigation Inc. The results are presented in Appendix D. They show the strength correlation factor, R_f, with values ranging from 0.31 to 0.76, hyperbolic exponent factor, n, with values ranging from 0.2 to 0.69, and the modulus constant, K, with values ranging from 223 to 1,232. The modulus constant values seemed low for soil with such high density. Considering the fact that the stiffness of a soil increases significantly with prestressing from loads such as compaction and the authors' personal experience, the stress-strain parameters used in the analyses are shown in Table 5.

Concrete properties

75. The concrete lock walls are assumed to behave linearly elastic in the finite element analyses in this study. The primary properties of the concrete needed for these analyses are the modulus of elasticity, Poisson's ratio, and unit weight. The most important of these is the modulus of elasticity. The modulus is the primary parameter controlling the magnitude of the displacement of the wall. In Part III, the test results show that modulus value decrease from the top of the wall to the bottom. The value ranged from 3.5 to 7.7 X 10^6 psi with a mean value of 6.08 X 10^6 psi and a standard deviation of 1.20 X 10^6 psi (approximately 20 percent). Having a lower value of the modulus in the bottom 25 ft of wall had less than a 5 percent change in the stresses and less than a 3 percent difference in the displacements for the Eisenhower north wall model. These differences are not significant enough to justify using a

varying modulus for the concrete walls. Therefore, a value of 5.0 X 10^6 psi was used in the analyses. The unit weight of the concrete ranged from 153.7 pcf to 163.8 pcf. A value of 150 pcf was used. The lower than average values for the elastic modulus and unit weight used in the analysis were chosen because concern over the influence of large aggregate size on biasing the average values. Poisson's ratio values ranged from 0.16 to 0.30. A value of 0.25 was used. The tensile strength was needed in the analyses to determine the stress level where cracking would occur. The laboratory tests showed a range for the splitting tensile strength from a value below 400 psi to values over 700 psi. The tensile stress range assumed for initiation of a crack was 300 to 400 psi.

Analysis Results

76. Since the results for the three monoliths show the same general trends and approximately the same maximum and minimum stress levels, only the results for the north wall at Eisenhower Lock will be discussed in detail. The results for the other two monoliths are presented in Appendixes E and F.

Initial stress state after dewatering and excavation

77. The first important step in the analysis was to establish the initial state of stress in the natural soil after the excavation for construction of the lock and in the lock walls after placement of the concrete. This was accomplished in two steps; (1) placement of the natural soil at the configuration of the excavation and (2) placement of the wall. Figure 28 shows a plot of the mobilized shear strength of the natural soil. Figure 29 shows the distribution of vertical stresses in the lock wall after its placement. Figure 30 shows the distribution of the minor principal stress in the wall after its placement. When minor principal stresses in an element exceed the limiting value of negative 57,600 psf (400 psi) or a zone of element has stresses greater than 43,200 psf (300 psi), a crack is assumed to occur. Negative values denote tensile stresses, while positive values denote compression.

Placement of backfill

78. Figures 31 and 32 show the contours of the vertical and horizontal displacement, respectively, resulting from the placement of the backfill prior to the establishment of the crack at the corner of landward ceiling of the filling/emptying culvert. The greatest vertical displacement is 0.29 ft and is located on the boundary between the backfill and natural soil. As found by Clough and Duncan (1969), the maximum displacement is located midway in the soil stratum being placed. The maximum vertical displacement in the wall occurs near the top of the wall and is less than 0.04 ft. The maximum horizontal displacement is 0.12 ft and is located at the top of the chamber wall. These displacements occur during the placement of the backfill and to our knowledge no measurements are available for comparison. Figure 33 shows mobilized shear strength for the soil after backfill placement. The shear strength in the backfill is fully mobilized near the top of the wall. This is expected because the wall is moving away from the backfill initiating the beginning of the development of active earth pressures.

79. Figure 34 shows the distribution of the minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of the backfill. Figure 35 shows that the value of the minor principal stress near the landward corner of the ceiling of the culvert has exceeded the limiting value for initiation of a crack. Figure 35 shows that one element has exceeded the 57,600 psf (400 psi) level and Figure 36 shows a zone surrounding the landward ceiling corner with stresses greater than 42,000 psf (290 psi). The differences in stress levels shown in Figures 35 and 36 is due to the contouring algorithm in the plotting software. In Figures 35 and 36, the initiation of the crack can be seen. Figure 37 shows the distribution of the major principai stresses and the deformed shape of the wall and surrounding backfill after placement of the backfill. The maximum compressive stresses ranging from 120,000 (833 psi) to 110,000 psf (764 psi) are found on the chamber face of the wall at the change in slope near the toe and surrounding the chamber side of the ceiling corner of the culvert. Figure 38 shows the major principal stresses surrounding the landward ceiling corner of the culvert. The major principal stresses at the corner are approaching zero. This analyses was used to establish the lateral earth pressure

required to cause the existing crack at the landward ceiling corner of the filling and emptying culvert.

80. Once the crack has formed, zero tensile strength and stiffness is provided across the crack and stresses carried across prior to the formation of the crack are redistributed into other portions of the wall. Figures 39 and 40 show the distribution of vertical and horizontal displacements, respectively. The displacements are essentially the same with or without the crack. Figure 41 shows the distribution of the minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of the backfill for this case. In Figure 42, a close-up of the culvert region is shown. From this figure, the opening of the crack can be seen. A gap to the back-filled face of the wall is not seen in this figure because the computed displacements are so small, but the crack is open to the backfill face of the wall. Comparing the minor principal stress distribution in Figure 41 with that in Figure 34 shows that the tensile stresses once carried across the crack are redistributed to the concrete along the chamber side of the culvert and to the concrete below the culvert. Before cracking, the minor principal stress in these regions ranged from approximate 0 to -12,000 psf (83.33 psi). After the formation of the crack the minor principal stresses increased in tension in these regions to a range from -4,000 psf (27.8 psi) to -16,000 psf (111.1 psi). Figure 43 shows the distribution of the major principal stresses and the deformed shape of the wall and surrounding backfill after placement of the backfill for this case. The maximum compressive stresses ranging from 120,000 psf (838 psi) to 108,000 psf (750 psi) are found on the chamber face of the wall at the change in slope near the toe and surrounding the chamber side of the ceiling corner of the culvert. These stress levels and locations are approximately the same as seen in Figure 37. Figure 44 shows the major principal stresses surrounding the landward ceiling corner of the culvert. The major principal stresses at the corner are approaching zero.

81. Figure 45 show the resulting lateral earth pressures acting on a vertical plane directly over the heel and the pressuremeter results from Gannet Fleming Engineering's investigation (1986). The predicted lateral earth pressures show good agreement with the pressuremeter results in the upper portion of the backfill. In the

mid height range, predicted values are higher than the pressuremeter results. In the lower portion of the backfill, there were no pressuremeter results for comparison. Figure 46 shows the shear stress from the finite element analysis on the vertical plane above the heel. The shear stress in the upper portion of the backfill relates to a wall friction of 8 to 10 deg and in the lower portion of the backfill, it relates to a wall friction of 28 to 30 deg.

Placement of the anchors

82. Figure 47 shows contours of the horizontal displacements. It shows that the post-tensioning of the anchor moves the top of the wall back into the backfill by 0.003 ft. This is an expect result of the post-tensioning of the anchors. Figure 48 show the distribution of the minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of the anchors. It shows a slight increase in the compressive stresses on the face of the chamber and surrounding the culvert. Also, there is a change in the stresses at the anchor head. Figure 49 shows the distribution of the minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of the anchors. The distribution of stresses is only changed in the top of the wall with a slight reduction in the minimum value of the tensile stresses, 26,390 psf (183 psi) to 24,610 psf (171 psi). Figure 50 shows a close-up near the culvert. The crack has partially closed with the placement of the anchors. This is consistent with the leakage found in the field. The leakage suggests that the crack is not completely closed.

Lower pool

83. Figure 51 shows the distribution of horizontal displacements. It shows that the top of wall moved 0.001 ft toward the backfill due to the raising of the water in the lock chamber to the lower level. Figures 52 and 53 show the distribution of the major and minor principal stresses, respectively, and the deformed shape of the wall and surrounding backfill after raising the pool to el 200 ft. The distribution of stresses shows a slight reduction in the maximum and minimum stress levels. These results show that this load case is less critical than the dewatered case.

Upper pool

84. Figure 54 shows the distribution of horizontal displacements. It shows that the top of wall moved 0.009 ft toward the backfill due to the raising of the water in the lock chamber to the lower level. Figures 55 and 56 show the distribution of the major and minor principal stresses, respectively, and the deformed shape of the wall and surrounding backfill after raising the pool to el 240 ft. The distribution of stresses shows a slight reduction in the maximum and minimum stress levels.

Summary and Conclusions

85. The static stress analyses have shown that the most critical loading condition is when the lock is completely dewatered. For this condition, the static analyses have shown that the chamber lock walls were overstressed in the past, due most likely to over compaction of backfill and the 20 ft higher than anticipated water level in the backfill. This overstressing was sufficient to cause the present crack from the upper landward corner of the filling/emptying culvert and extending to the surface of wall on the landward. Once the crack had occurred and the anchors were placed (the current state of the walls), the analyses show that the tensile stress that caused the crack to form has been redistributed to the concrete along the chamber side of the culvert and to the section below the culvert. This redistribution of tensile stresses has not overstressed the lock walls and leaves them with a factor safety greater than 2.0 (tensile stress of 180 psi from analyses versus 400 psi cracking limit) against tensile stress induced cracking. The compressive stresses are well below (3.5 times) the lowest compressive strength found in the laboratory test program. The location of the highest compressive strength is in an area where the concrete deterioration has been a problem.

Location	Soil Type	Unit Weight (lbs/ft)	Strength Parameters		Hyperbolic Parameters				
			C (psf)	ϕ (degrees)	K	n	K_{ur}	ν	R_f
Backfill	Compacted Glacial Till	148	500	45	1800		2000	0.35	
Natural Soil	Glacial Till	135	500	38	1200		1500	0.35	

EQUATIONS FOR SOIL STRESS-STRAIN MODEL

$$E_i = KP_A\left(\frac{\sigma_3}{P_A}\right)^n$$

$$SL = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_{FAILURE}}$$

$$E_t = E_i(1 - R_f\, SL)^2$$

$$E_{UR} = K_{UR}P_A\left(\frac{\sigma_3}{P_A}\right)^n$$

P_A = atmospheric pressure

Table 5. Soil Model Parameters

PART V: SEISMIC ANALYSIS OF THE NORTH WALL OF THE EISENHOWER LOCK INCLUDING FLUID-SOIL-STRUCTURE INTERACTION

Introduction and Objectives

86. The active earth pressures and water pressures due to a horizontal seismic motion on the lock have been estimated for the Eisenhower Project Site. Response spectra from the Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE) were used to perform the linear dynamic stress analysis. Two dimensional plane strain analysis was used to estimate earth pressures and water pressures. These pressures were then added to the dynamic results to yield the final stress values. Figure 57 shows a typical section of the north wall of the lock.

87. The purpose of the present report is to use current analysis methods, which could incorporate the effects of soil structure interaction for a seismic analysis of the lock. In addition, the report provides pertinent state-of-the-art information on soil-structure interaction and identifies critical areas of the lock structure that should be carefully engineered.

General Overview of Analysis

88. Retaining walls designed for earthquake effects may be grouped into two main categories on the basis of their failure mode:

- a. Gravity retaining walls, using the mass of the wall for stability, which fail by sliding or by overturning. These may be called displacement-governed walls. The internal force levels in such walls are of secondary importance.
- b. Cantilever, tied-back or braced walls, which fail by fracture or yield of one of the structural components involved. These walls may be called force-governed walls. For at least some part of such structural systems the internal force level is important.

The present analysis assumes that no displacement failure is anticipated, and the Eisenhower Lock is composed of force-governed walls.

89. Generally the objectives in earthquake-resistant design are as follows:

a. To ensure that no significant structural damage results from a moderate earthquake having a reasonable likelihood of not being exceeded during the life of the structure.

b. To ensure against collapse or major structural failure for a rare severe earthquake.

90. The first design objective can be met by sizing the structural members such that their strengths will not be significantly exceeded in an earthquake of moderate intensity. At this design level, the behavior is essentially linear elastic and application of elastic response spectrum analysis can be used to arrive at critical member forces.

91. The second design objective of ensuring against a major structural failure is often met by employing design details that ensure ductile behavior.

92. These two objectives can be met if the necessary steel reinforcement is provided. This is always the case for residential buildings and a certain class of lifeline structures such as bridges. For retaining walls that are very lightly reinforced however, the objectives are still the same, but the objectives are met in a somewhat different way.

93. The "dual spectrum approach" is used if two distinct levels of prescribed earthquake excitations are employed to meet the two aforementioned design objectives. The higher intensity motion is often called "maximum credible earthquake" (MCE) or ductility level earthquake and the lower level motion is referred to as "operating basis earthquake" or strength level earthquake. The MCE spectrum reflects an envelope of response values which have a high probability (e.g., 85-95 percent) of not being exceeded during the lifetime of the structure. The OBE spectrum has a moderate probability (e.g., 50-60 percent) of not being exceeded. The structure should not suffer any damage from the OBE. Damage is acceptable for the MCE, but the structure should remain mostly intact and not experience a catastrophic failure.

94. For gravity retaining walls and in particular for forced-governed walls, the members are sized so that failure is avoided and only reparable damage is allowed to develop. This is accomplished by sizing the wall sections so that the strength of the contraction material will be exceeded in limited areas that retain the function of the structure and are accessible to retrofiting after the earthquake.

95. The analysis reflects the "dual approach." The procedure for both the OBE and MCE is the same with the only difference being the intensity of the lateral forces acting on the lock wall.

96. The following steps comprise the analysis:

a. Evaluate lateral stiffness of soil in contact with the wall.

b. Evaluate earth pressures on the wall.

c. Stress analysis using the OBE and MCE loading conditions.

The dynamic analysis of the wall was performed with ADINA (ADINA 1988).

97. The wall was modeled with plane strain rectangular elements, while spring elements were used to represent the soil. The evaluation of the spring constants is based on state-of-the-art soil dynamics procedures. The minor differences of the frictional angle of the soil, and the modulus of elasticity of the concrete E_c, used in the static analysis and the dynamic analysis are insignificant given the complexity of the system and the computational procedures in both analyses. The finite element method (FEM) modelling of the lock, i.e., size of elements, and mesh were performed according to guidelines found in FEM literature (Bathe 1982). The FEM model used to extract the mode shapes as well as performing the response spectra analysis is shown in Figure 58.

98. The analysis revealed that the soil-structure interaction (SSI) does not effect the dynamic response of the lock significantly - at least the effects on the natural frequency of the fundamental mode shape. Without the soil, the natural frequency was 12.1 Hz; with the soil, the natural frequency was 12.7 Hz, for a total percent difference of 4.7 percent.

99. As a consequence of this small change, the earth pressures acting on the wall have been changed insignificantly from the ones evaluated without soil-structure interaction.

100. It should be pointed out that SSI, as has been accounted for in this analysis, is a rough approximation of SSI behavior under seismic loads. Salient deficiencies of the analysis include:

a. Evaluation of the soil spring stiffness from a shear modulus, G_s, estimated from earthquake induced strains.

b. Assumption that the soil stiffness is constant. This is not always a conservative assumption since the magnitude of the soil stiffness is highly dependent on the frequency of the excitation (Wolf 1985).

c. Neglect of radiation damping in the soil. Radiation damping is a measure of the energy loss from the structure through radiation of waves away from the footing, i.e., it is a purely geometrical effect. For horizontal and vertical translations, radiation damping may be large (greater than 10 percent of critical damping).

Analysis

101. The seismic analysis used the response spectra method. Two cases were considered:

a. OBE with 5 percent damping.

b. MCE with 10 percent damping.

The 10 percent damping for the MCE is an approximation that accounts for structural damage. This is a standard method used in response spectra and thus linear for seismic analysis.

102. The response spectra for both the OBE and MCE is an envelope of maximum values of the site specific response spectra provided by Krinitzsky (1991). The response spectra are for a Modified Mercalli intensity VIII earthquake. The time histories and response spectra are for a near field, hard site, and shallow earthquake. The response spectra used are shown in Figure 59.

103. A total of 10 mode shapes were used to compute the seismic stresses. The method used a standard response spectra method to combine the stresses from each mode shape to calculate the net stresses. This method was a square root sum of squares (SRSS) method, thus all stress values are positive. Although this is confusing at first, it causes no problems in interpreting the results. Since it is an earthquake loading and thus a cyclic loading, the stresses are equally likely to be tensile or compressive. Simply adding the dynamic stress to the static stresses will give the largest tensile stresses which are important to determine if failure is occurring.

104. The static stresses are from the SSI calculations covered previously in this report. Note that the sign convention for the stresses are different. Tensile

stresses are positive while compressive stresses are negative. Also, note that all dimensions and elevations are given in inches and stresses are in pounds per square inch.

Results

105. The results from the dynamic analysis are shown starting with Figure 60. This figure is a contour plot of the maximum principal stresses for the OBE loading. The largest stress value is 300 psi located at el 2,678 on the exterior or right face of the lock. This stress is well within the tensile strength of the concrete (f_t=500 psi).

106. The results for the MCE are shown in Figure 61. The maximum stress value is in the same location as is the OBE. Now, the maximum principal stress is 405 psi. This is getting close to the tensile strength of the concrete.

107. Combining the OBE stresses with the static stresses changes the maximum stress value very little. The OBE combined with the static load case is shown in Figure 62. The combination did create another high stress region near the base of the right face. This new stress is 225 psi.

108. Figure 63 shows the MCE stresses combined with the static stresses. Again, very little change occurs in the location or magnitude of the maximum stress value. It is at el 2,678 on the exterior face of the lock. The stress near the base of the right face is now 250 psi.

109. For this structure, since the maximum dynamic stresses occur near the top of the structure, which is a low stress area for the static load case, and since the static stresses are high in a region where the dynamic stresses are low, the combination of static and dynamic stresses does not generate very high stress levels.

Conclusions

110. For both the OBE and MCE conditions, catastrophic failure of the lock wall is unlikely to occur. No damage should occur during the OBE event with a maximum principal stress of 300 psi. There is a possibility of damage to the structure

during the MCE event near the top of the structure where the sloping face transitions into a vertical face. If damage does occur during the MCE event, no more than about 5 to 10 percent of the section would be damaged.

PART VI: CONCLUSIONS

111. For static loading and with the current condition as analyzed, the chamber wall monoliths at Eisenhower and Snell Locks show no evidence of being over-stressed. The results of the static analyses show that the chamber wall monoliths have tensile stresses below 200 psi and compressive stresses less than 1,000 psi. These stress levels are below any of the strengths determined from the laboratory testing program (390 psi for tensile splitting tests and 3,660 psi for compression tests). The highest stresses under static loading are in areas of low strength as determined in the testing program. If for some reason the cross sectional area is significantly reduced, such as concrete deterioration as found in the history of Eisenhower Lock, the structural integrity would be in question. It is important for the SLSDC to maintain an aggressive maintenance program of replacing deteriorated concrete. Immediate attention should be given to the repair of deteriorated concrete near the bottom of the lock walls at Eisenhower Lock.

112. For seismic loading events, catastrophic failure of the lock walls is unlikely. No damage should occur during the OBE event where combined static and dynamic tensile stresses reaches 300 psi. There is a possibility of some damage to the walls during the MCE event near the top of the structure where the sloping face transitions into a vertical face. If damage does occur during the MCE event, no more than about 5 to 10 percent of the section would be damaged. Where the MCE event has a low likelihood of occurrence and the damage level is low, it would be prudent to wait and make repairs to the chamber walls after such an earthquake event.

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Table 1						
Lock	Boring No.	Elevation Top of Boring, ft.	Elevation Top of Rock, ft	Concrete Bonded to Rock	Elevation Bottom of Boring	Boring Depth, ft.
Eisenhower	N-53	251.5	141.1	No	120.4	131.1
Eisenhower	N-57	251.5	138.9	No	126.5	125.0
Eisenhower	S-17	251.5	141.9	Yes	128.7	122.8
Eisenhower	S-19	251.5	140.8	No	122.5	129.0
Eisenhower	SS-18	251.5	*	NA	215.7	35.8
Eisenhower	SS-19	251.5	*	NA	216.5	35.0
Snell	N-56	205.0	*	NA	131.1	73.9
Snell	S-15	205.0	105.9	No	100.7	104.3

* Boring did not extend to concrete rock interface.
NA - Not applicable.

Table 2. Visual Inspection of Cores for Eisenhower and Snell Locks				
Hole	Box No.	Specimen No.	Concrete or Rock	Comments
N-53	4	38	Concrete	Dense, well consolidated
		39	Concrete	Dense, well consolidated
	10	40	Concrete	Minor voids; poor agg/paste interface
		41	Concrete	Minor voids
	19	42	Concrete	Considerable voids; poor interface; chipped end
		43	Concrete	Poor consolidation; broke while sawing
	25	44	Concrete	Minor voids; poor agg/paste interface
		45	Concrete	Considerable voids
	26	46	Concrete	Minor voids
		47	Concrete	Considerable voids; poor agg/paste interface
	28	48	Rock	Sound, intact
		49	Rock	Sound, intact
	19	43R	Concrete	Minor voids; poor agg/paste interface
N-57	7	1	Concrete	Broke in half while sawing
		2	Concrete	Minor voids
	15	3	Concrete	Minor voids; poor interface
		4	Concrete	Minor voids
	19	5	Concrete	Minor voids; chipped end; poor interface
		6	Concrete	Considerable voids; chipped end
	25	7	Concrete	Considerable voids; poor agg/paste interface; big rock
		8	Concrete	Considerable voids; poor interface

Table 2. (Continued)				
Hole	Box No.	Specimen No.	Concrete or Rock	Comments
	26	9	Concrete	Minor voids; poor interface; chipped end
		10	Concrete	Considerable voids
	29	11	Rock	Sound, intact
		12	Rock	Sound, intact
	7	1R	Concrete	Minor voids: poor agg/paste interface; big rock
S-17	7	13	Concrete	Minor voids; one big rock in specimen
		14	Concrete	Minor voids
	10	15	Concrete	Minor voids
		16	Concrete	Minor voids
	17	17	Concrete	Minor voids; #5 steel anchor in specimen
		18	Concrete	Minor voids; poor agg/paste inter-face
	24	19	Concrete	Minor voids
		20	Concrete	Minor voids
	26	21	Concrete	Minor voids
		22	Concrete	Minor voids; poor interface; big rock
	27	23	Both	Good bond @ C/R interface
	29	24	Rock	Sound, intact
		25	Rock	Sound, intact

Table 2. (Continued)				
Hole	Box No.	Specimen No.	Concrete or Rock	Comments
S-19	1	26	Concrete	Minor voids; poor interface
		27	Concrete	Minor voids; poor interface; chipped end; big rock
	11	28	Concrete	Minor voids; chipped end
		29	Concrete	Minor voids; chipped end
	16	30	Concrete	Considerable voids
		31	Concrete	Minor voids
		32	Concrete	Minor voids
		33	Concrete	Minor voids; big rock
		34	Concrete	Minor voids; big rock
		35	Concrete	Minor voids; poor interface; big rock
N-56	3	62	Concrete	Dense, well consolidated
		63	Concrete	Dense, well consolidated
	9	64	Concrete	Minor voids
		65	Concrete	Minor voids
	12	66	Concrete	Minor voids
		67	Concrete	Minor voids
	16	68	Concrete	Minor voids
		69	Concrete	Minor voids
	17	70	Concrete	Minor voids
		71	Concrete	Minor voids; chipped end

Table 2. (Concluded)				
Hole	Box No.	Specimen No.	Concrete or Rock	Comments
S-15	1	50	Concrete	Minor voids
		51	Concrete	Minor voids
	6	52	Concrete	Minor voids; poor interface
		53	Concrete	Minor voids
	11	54	Concrete	Considerable voids; poor interface
		55	Concrete	Considerable voids; poor agg/paste interface
	19	56	Concrete	Minor voids
		57	Concrete	Minor voids
	23	58	Concrete	Minor voids
		59	Concrete	Minor voids
	24	60	Rock	Sound, intact
		61	Rock	Sound, intact

Table 3. Results of Tests on Cores from Eisenhower Lock, Hole N-53

Spec. No.	Core Run, ft		Concrete or Rock	Description	Density, pcf	Percent Voids	Pulse Velocity, fps	Compressive Strength, psi	Elastic Modulus, million psi	Poisson's Ratio	Splitting Tensile Strength psi
	From	To									
4-38	10.7	14.8	Concrete	Dense, well consolidated	163.8	--------	18942	6000	7.75	0.28	-------
4-39	10.7	14.8	Concrete	Dense, well consolidated	163.4	9.3	--------	--------	--------	--------	665
10-40	34.6	39.4	Concrete	Minor voids; poor agg/paste interface	158.5	--------	--------	--------	--------	--------	580
10-41	34.6	39.4	Concrete	Minor voids	159.6	11.7	18269	6060	6.00	0.23	--------
19-42	75.3	79.5	Concrete	Considerable voids; poor interface; chipped end	158.8	13.9	--------	--------	--------	--------	590
19-43R	75.3	79.5	Concrete	Poor consolidation; broke while sawing	158.9	--------	--------	--------	--------	--------	610
25-44	100.2	104.8	Concrete	Minor voids; poor agg/paste interface	157.8	8.9	--------	--------	--------	--------	760
25-45	100.2	104.8	Concrete	Considerable voids	154.2	7.8	16943	4430	3.50	0.16	--------
26-46	104.8	108.6	Concrete	Minor voids	157.6	--------	17824	3610	5.20	0.22	--------
26-47	104.8	108.6	Concrete	Considerable voids; poor agg/paste interface	157.0	9.7	--------	--------	--------	--------	525
28-48	112.0	116.0	Rock	Sound, intact	175.2	--------	21137	34810	14.50	0.31	--------
28-49	112.0	116.0	Rock	Sound, intact	172.7	--------	19841	41150	11.60	0.27	--------

Table 4. Results of Tests on Cores from Eisenhower Lock, Hole N-57

Spec. No.	Core Run, ft		Concrete or Rock	Description	Density, pcf	Percent Voids	Pulse Velocity, fps	Compressive Strength, psi	Elastic Modulus, million psi	Poisson's Ratio	Splitting Tensile Strength psi
	From	To									
7-1R	24.8	29.3	Concrete	Minor voids; poor agg/paste interface; big rock	160.4	10.5	--------	--------	--------	--------	520
7-2	24.8	29.3	Concrete	Minor voids	158.4	10.1	17381	4520	6.35	0.22	--------
15-3	58.8	63.3	Concrete	Minor voids; poor interface	160.2	10.6	17691	5450	5.75	0.21	--------
15-4	58.8	63.3	Concrete	Minor voids	159.9	--------	--------	--------	--------	--------	760
19-5	75.0	79.2	Concrete	Minor voids; chipped end; poor interface	161.3	7.8	19036	5750	6.20	0.21	--------
19-6	75.0	79.2	Concrete	Considerable voids; chipped end	157.9	9.9	--------	--------	--------	--------	470
25-7	102.0	106.5	Concrete	Considerable voids; poor agg/paste interface; big rock	154.2	--------	--------	--------	--------	--------	540
25-8	102.0	106.5	Concrete	Considerable voids; poor interface	153.5	12.1	17170	3880	5.40	0.23	--------
26-9	106.5	111.3	Concrete	Minor voids; poor interface; chipped end	156.5	11.9	--------	--------	--------	--------	390
26-10	106.5	111.3	Concrete	Considerable voids	153.5	10.2	16854	4640	5.55	0.25	--------
29-11	119.0	122.8	Rock	Sound, intact	--------	--------	20599	16200	12.45	--------	--------
29-12	119.0	122.8	Rock	Sound, intact	--------	--------	--------	--------	--------	--------	--------

Table 5. Results of Tests on Cores from Eisenhower Lock, Hole S-17

Spec. No.	Core Run, ft		Concrete or Rock	Description	Density, pcf	Percent Voids	Pulse Velocity, fps	Compressive Strength, psi	Elastic Modulus, million psi	Poisson's Ratio	Splitting Tensile Strength psi
	From	To									
7-13	25.1	29.4	Concrete	Minor voids; one big rock in specimen	161.0	5.3	--------	--------	--------	--------	635
7-14	25.1	29.4	Concrete	Minor voids	158.2	10.5	17990	5980	7.05	0.26	--------
10-15	37.3	42.3	Concrete	Minor voids	158.7	10.6	--------	--------	--------	--------	645
10-16	37.3	42.3	Concrete	Minor voids	156.7	9.5	17183	4490	6.10	0.20	--------
17-17	67.5	71.9	Concrete	Minor voids; #5 steel anchor in specimen	161.4	9.4	17509	7280	5.35	0.23	--------
17-18	67.5	71.9	Concrete	Minor voids; poor agg/paste interface	156.6	9.1	--------	--------	--------	--------	410
24-19	95.1	99.7	Concrete	Minor voids	156.5	--------	17816	6150	5.55	0.18	--------
24-20	95.1	99.7	Concrete	Minor voids	155.7	11.1	--------	--------	--------	--------	790
26-21	103.8	108.0	Concrete	Minor voids	156.4	--------	--------	--------	--------	--------	555
26-22	103.8	108.0	Concrete	Minor voids; poor interface; big rock	155.0	11.6	15975	3750	3.70	0.17	--------
27-23	108.0	111.6	Both	Good bond @ C/R interface	--------	--------	--------	--------	--------	--------	--------
29-24	116.2	120.4	Rock	Sound, intact	--------	--------	20113	40280	12.25	0.30	--------
29-25	116.2	120.4	Rock	Sound, intact	--------	--------	19950	17010	11.90	0.25	--------

Table 6. Results of Tests on Cores from Eisenhower Lock, Hole S-19

Spec. No.	Core Run, ft		Concrete or Rock	Description	Density, pcf	Percent Voids	Pulse Velocity, fps	Compressive Strength, psi	Elastic Modulus, million psi	Poisson's Ratio	Splitting Tensile Strength psi
	From	To									
1-26	0	5.0	Concrete	Minor voids; poor interface	155.7	8.8	17478	6100	7.35	0.27	--------
1-27	0	5.0	Concrete	Minor voids; poor interface; chipped end; big rock	153.7	10.6	--------	--------	--------	--------	570
11-28	44.4	48.4	Concrete	Minor voids; chipped end	158.8	12.6	--------	--------	--------	--------	555
11-29	44.4	48.4	Concrete	Minor voids; chipped end	158.1	--------	17569	5320	6.75	0.30	--------
16-30	65.6	69.6	Concrete	Considerable voids	154.7	12.0	--------	--------	--------	--------	505
16-31	65.6	69.6	Concrete	Minor voids	155.1	--------	16752	5900	5.10	0.24	--------
20-32	83.1	88.0	Concrete	Minor voids	160.8	--------	17965	5890	5.00	0.17	--------
20-33	83.1	88.0	Concrete	Minor voids; big rock	160.0	12.3	--------	--------	--------	--------	490
24-34	100.4	105.0	Concrete	Minor voids; big rock	155.8	9.4	--------	--------	--------	--------	650
24-35	100.4	105.0	Concrete	Minor voids; poor interface; big rock	158.4	11.8	16239	4090	4.60	0.29	--------
26.36	108.2	112.4	Both	No bond @ C/R interface	--------	--------	--------	--------	--------	--------	--------
30.37	124.7	129.0	Rock	Sound, intact	174.0	--------	9873	19700	6.20	0.27	--------

Table 7. Results of Tests on Cores from Snell Lock, Hole S-15

Spec. No.	Core Run, ft		Concrete or Rock	Description	Density, pcf	Percent Voids	Pulse Velocity, fps	Compressive Strength, psi	Elastic Modulus, million psi	Poisson's Ratio	Splitting Tensile Strength psi
	From	To									
1-50	0	4.5	Concrete	Minor voids	156.5	13.6	17362	7420	6.60	0.24	--------
1-51	23.9	28.0	Concrete	Minor voids	159.9	8.4	--------	--------	--------	--------	680
6-52	23.9	28.0	Concrete	Minor voids; poor interface	159.1	--------	17343	5940	6.95	0.25	--------
6-53	23.9	28.0	Concrete	Minor voids	159.6	7.2	--------	--------	--------	--------	625
11-54	45.9	50.4	Concrete	Considerable voids; poor interface	155.5	--------	17686	7460	7.70	0.25	--------
11-55	45.9	50.4	Concrete	Considerable voids; poor agg/paste interface	155.3	17.8	--------	--------	--------	--------	635
19-56	80.5	84.5	Concrete	Minor voids	157.4	--------	18032	7870	6.40	0.23	--------
19-57	80.5	84.5	Concrete	Minor voids	158.6	11.5	--------	--------	--------	--------	520
23-58	96.2	100.2	Concrete	Minor voids	155.9	9.0	17046	3730	4.60	0.20	--------
23-59	96.2	100.2	Concrete	Minor voids	160.8	14.1	--------	--------	--------	--------	495
24-60	100.2	104.2	Rock	Sound, intact	--------	--------	16219	21700	8.05	0.27	--------
24-61	100.2	104.2	Rock	Sound, intact	--------	--------	16797	22560	9.05	0.34	--------

Table 8. Results of Tests on Cores from Snell Lock, Hole N-56

Spec. No.	Core Run, ft		Concrete or Rock	Description	Density, pcf	Percent Voids	Pulse Velocity, fps	Compressive Strength, psi	Elastic Modulus, million psi	Poisson's Ratio	Splitting Tensile Strength psi
	From	To									
3-62	8.7	12.9	Concrete	Dense, well consolidated	164.3	6.6	--------	--------	--------	--------	875
3-63	8.7	12.9	Concrete	Dense, well consolidated	162.9	5.4	18931	8360	7.50	.25	--------
9-64	33.7	37.7	Concrete	Minor voids	161.1	--------	18370	5980	6.45	0.26	--------
9-65	33.7	37.7	Concrete	Minor voids	161.6	10.2	--------	--------	--------	--------	495
12-66	45.8	50.0	Concrete	Minor voids	158.0	--------	--------	--------	--------	--------	930
12-67	45.8	50.0	Concrete	Minor voids	158.7	9.1	17493	8590	8.95	0.28	--------
16-68	62.6	67.1	Concrete	Minor voids	150.7	16.4	--------	--------	--------	--------	530
16-69	62.6	67.1	Concrete	Minor voids	157.2	11.8	17883	4540	6.90	0.28	--------
17-70	67.1	71.6	Concrete	Minor voids	155.3	--------	17956	6260	6.25	0.25	--------
17-71	67.1	71.6	Concrete	Minor voids; chipped end	156.8	12.9	--------	--------	--------	--------	710

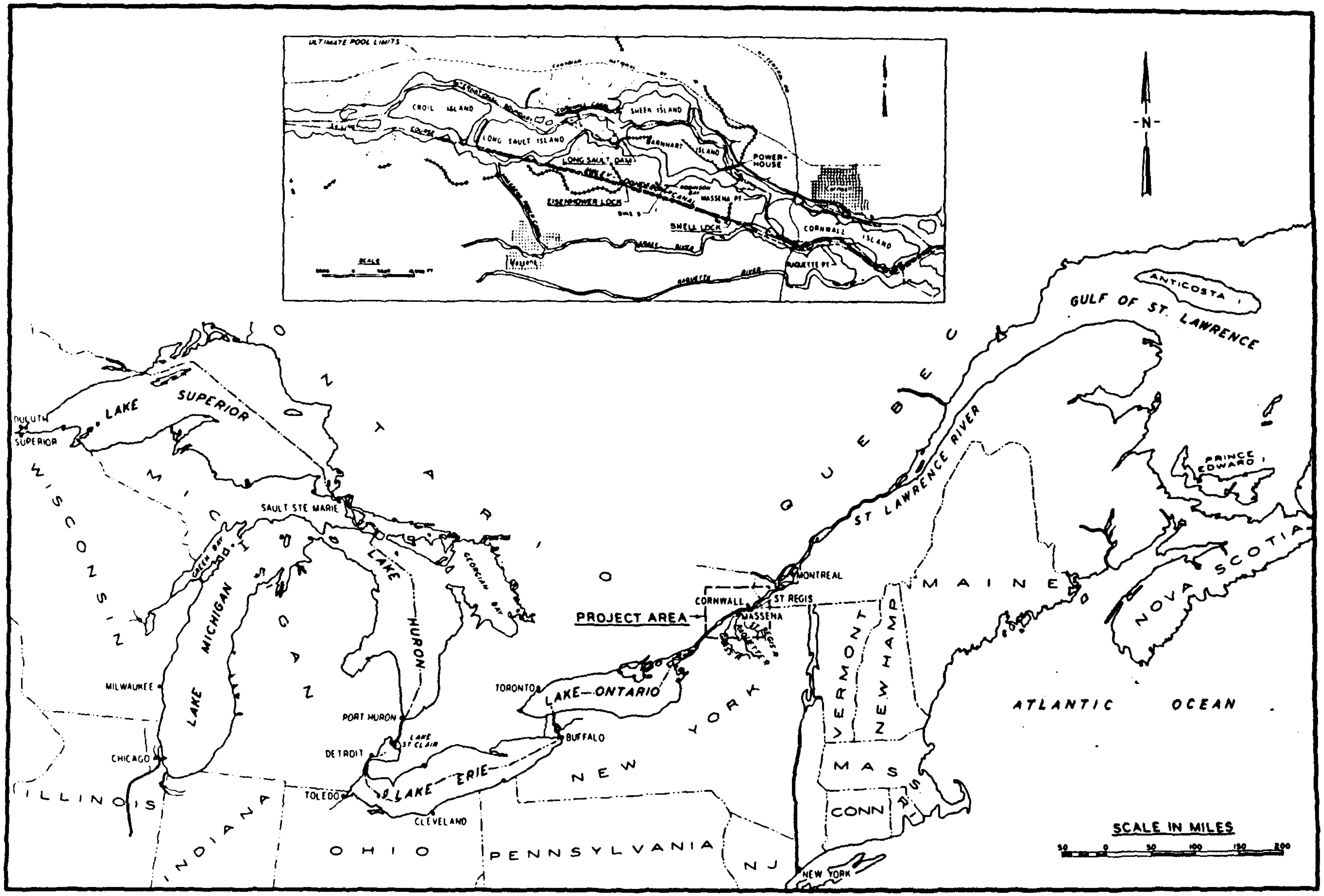


Figure 1. Vicinity map

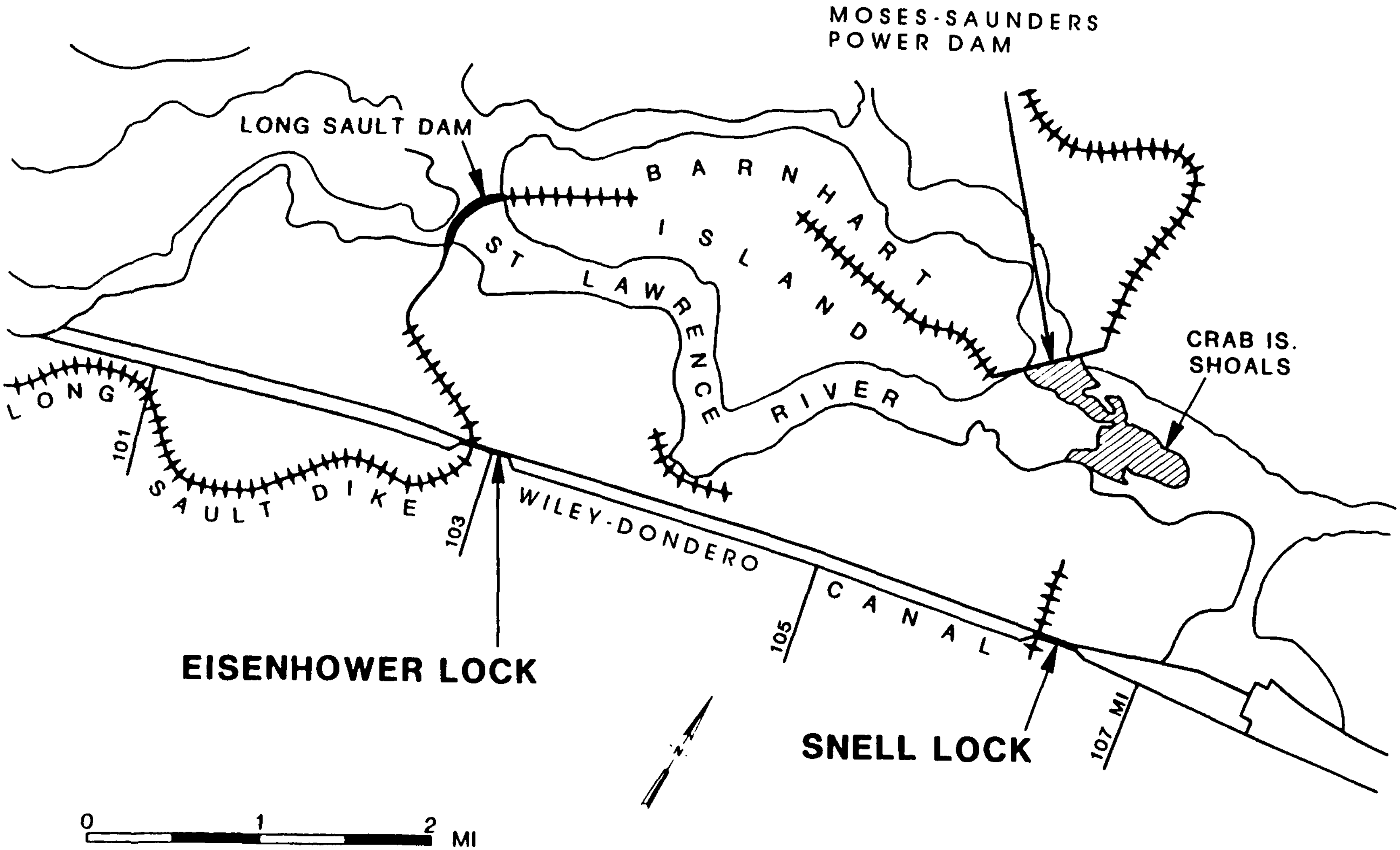


Figure 2. Location of Eisenhower and Snell Locks on the Wiley-Dondero Canal at Massena, New York

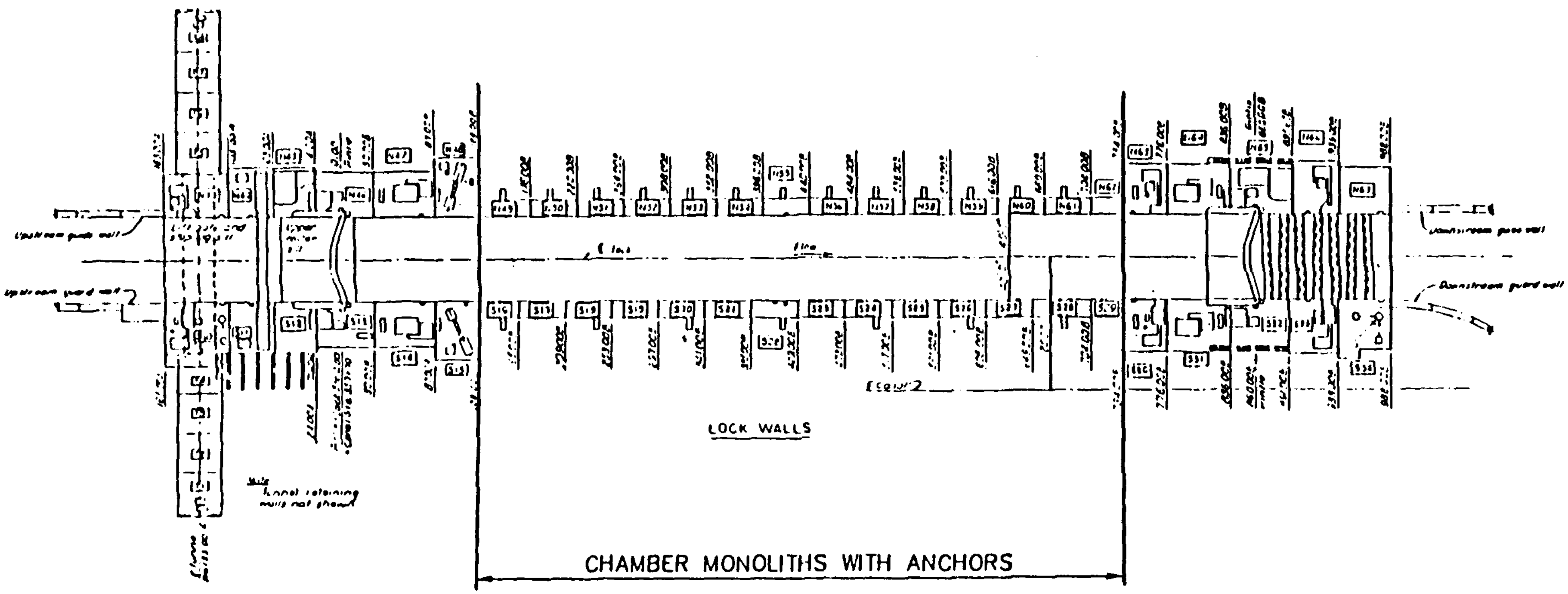


Figure 3. Plan view of Eisenhower Lock (not to scale)

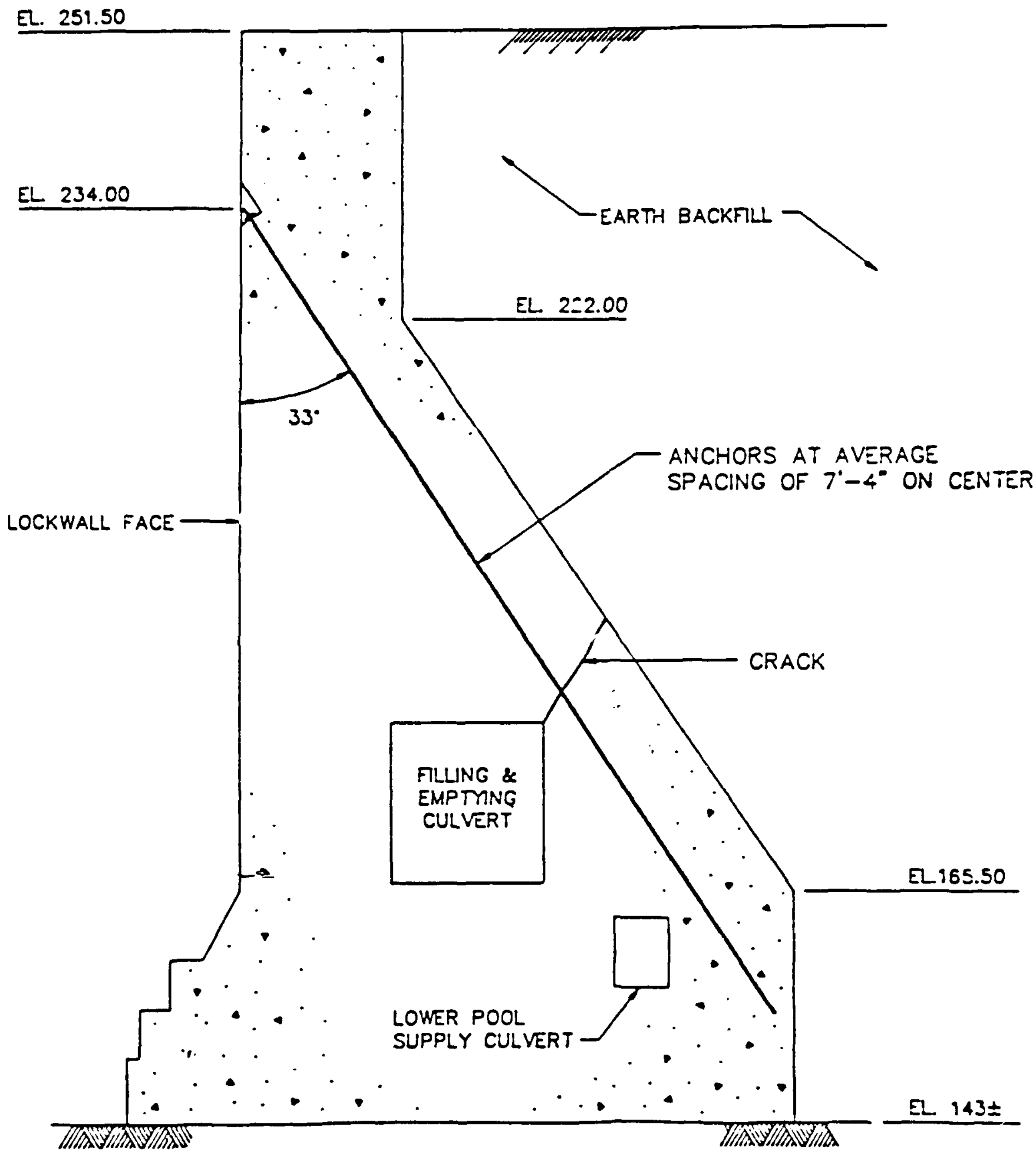


Figure 4. Typical section through south wall chamber monoliths (not to scale)

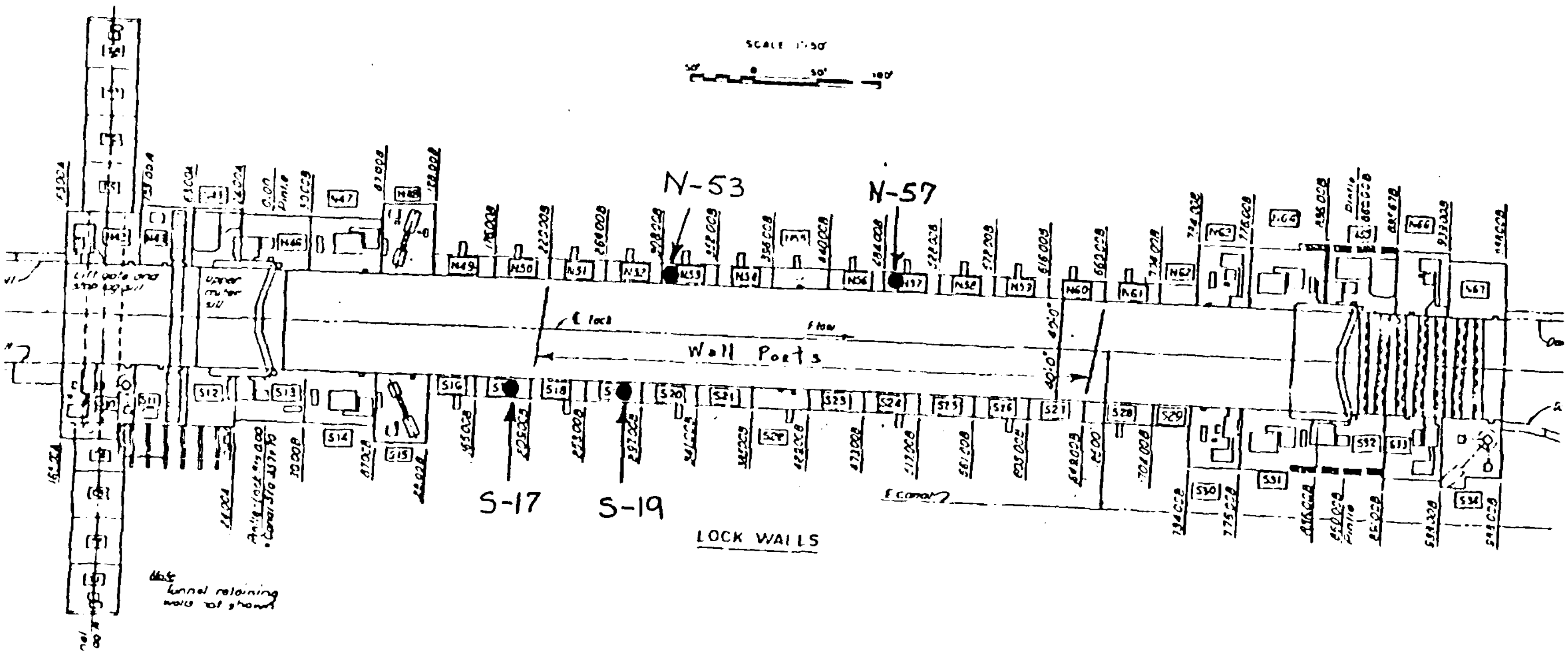


Figure 5. Boring locations at Eisenhower Lock

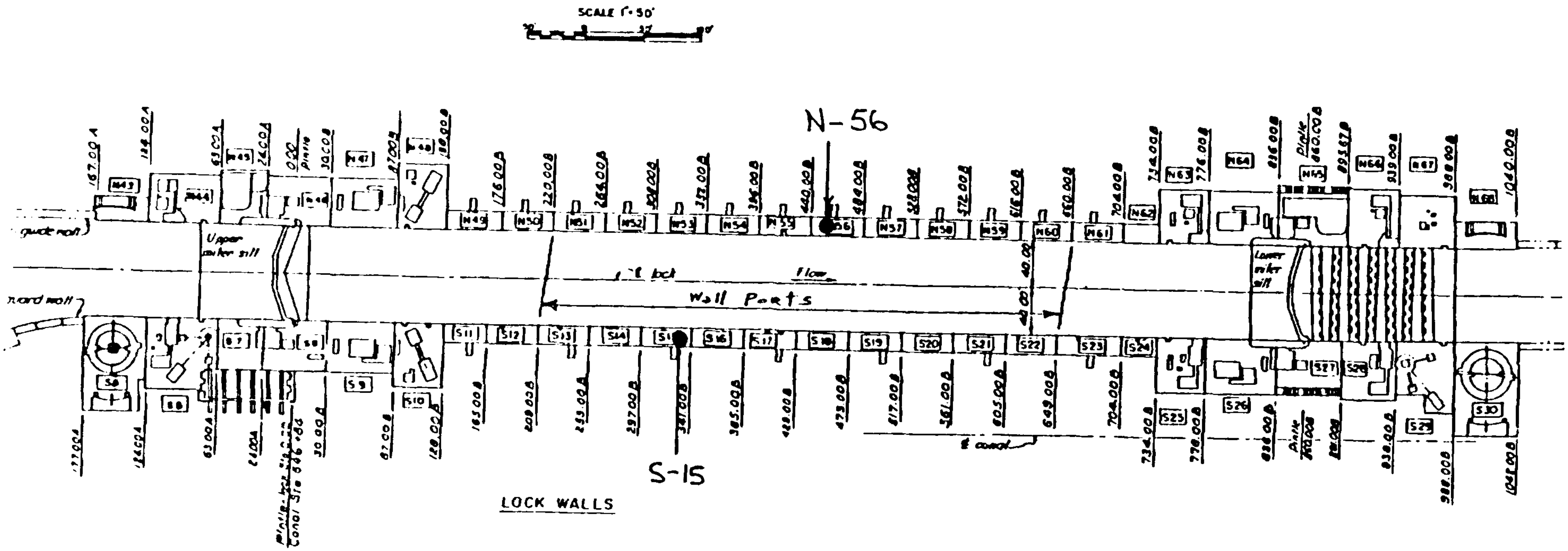


Figure 6. Boring locations at Snell Lock

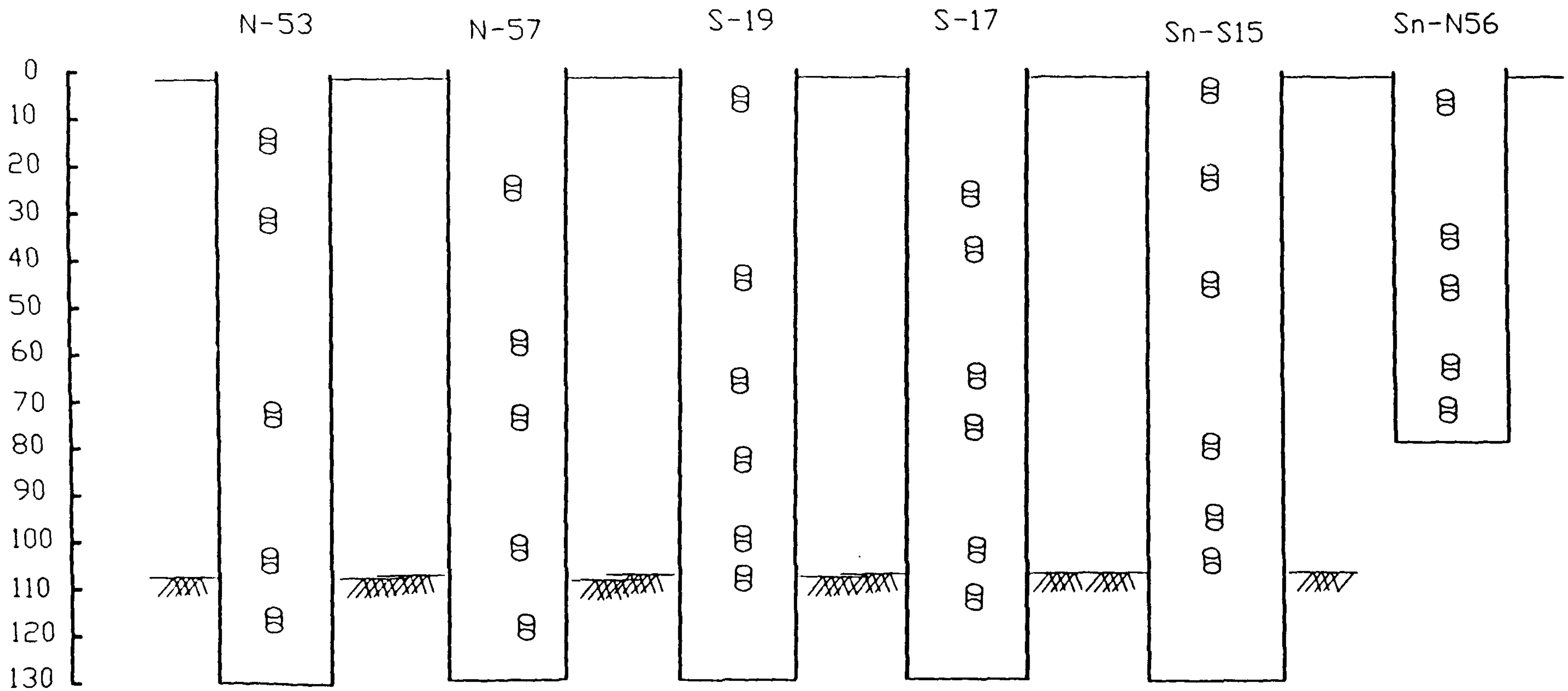


Figure 7. Schematic repesenation of locations of test samples

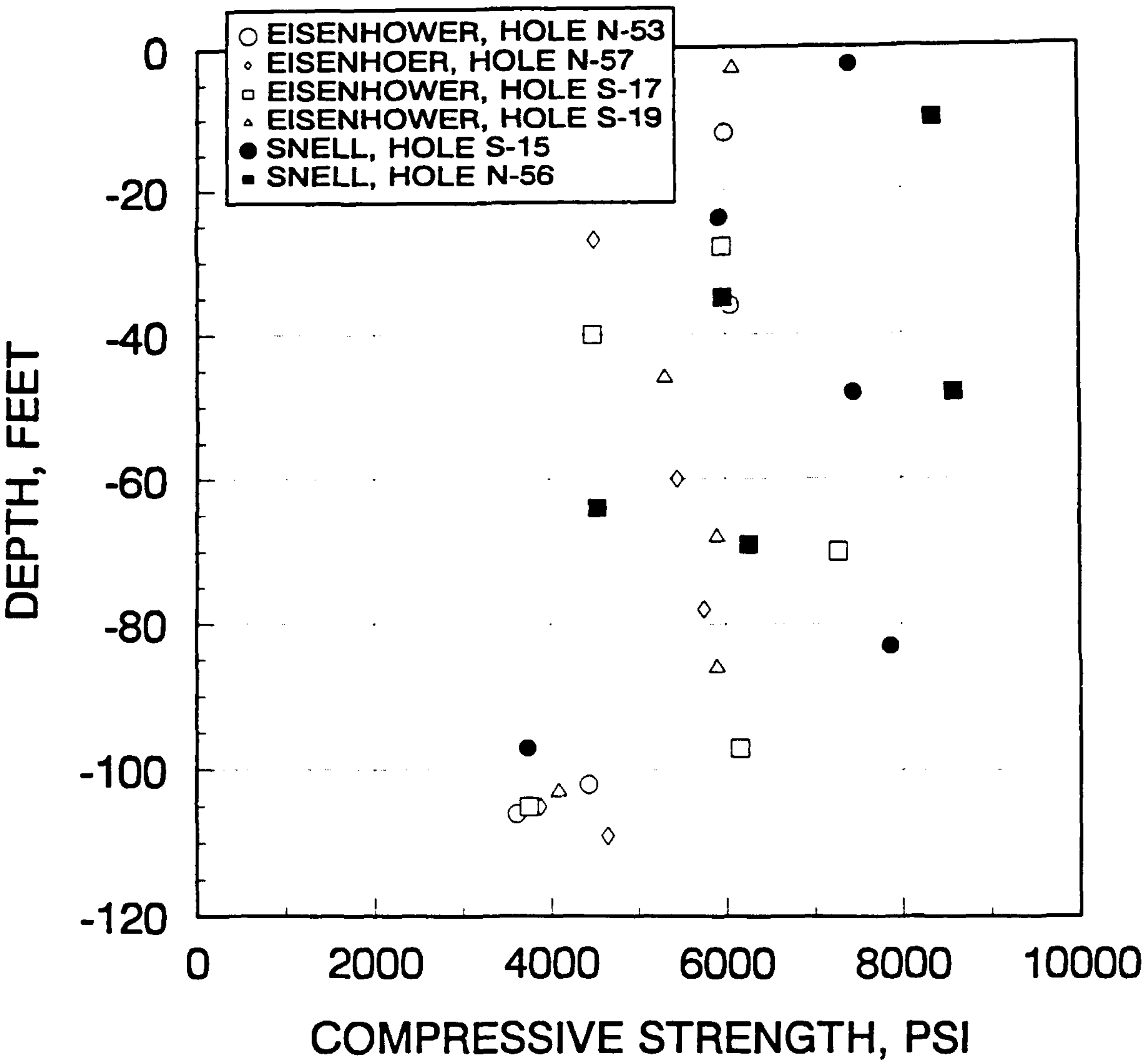


Figure 8. Variation of concrete compressive strengths with depth

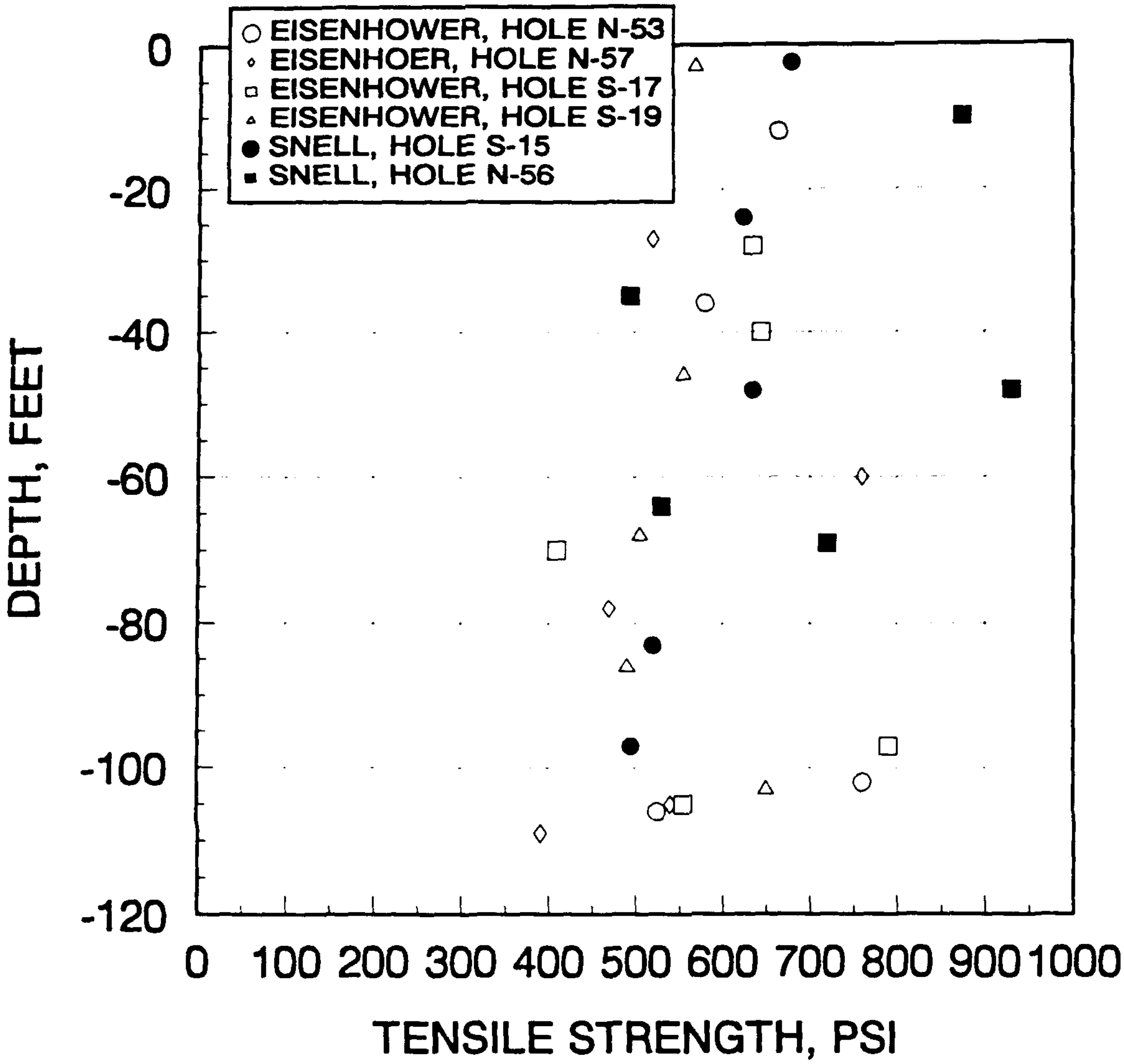


Figure 9. Variation of concrete tensile strengths with depth

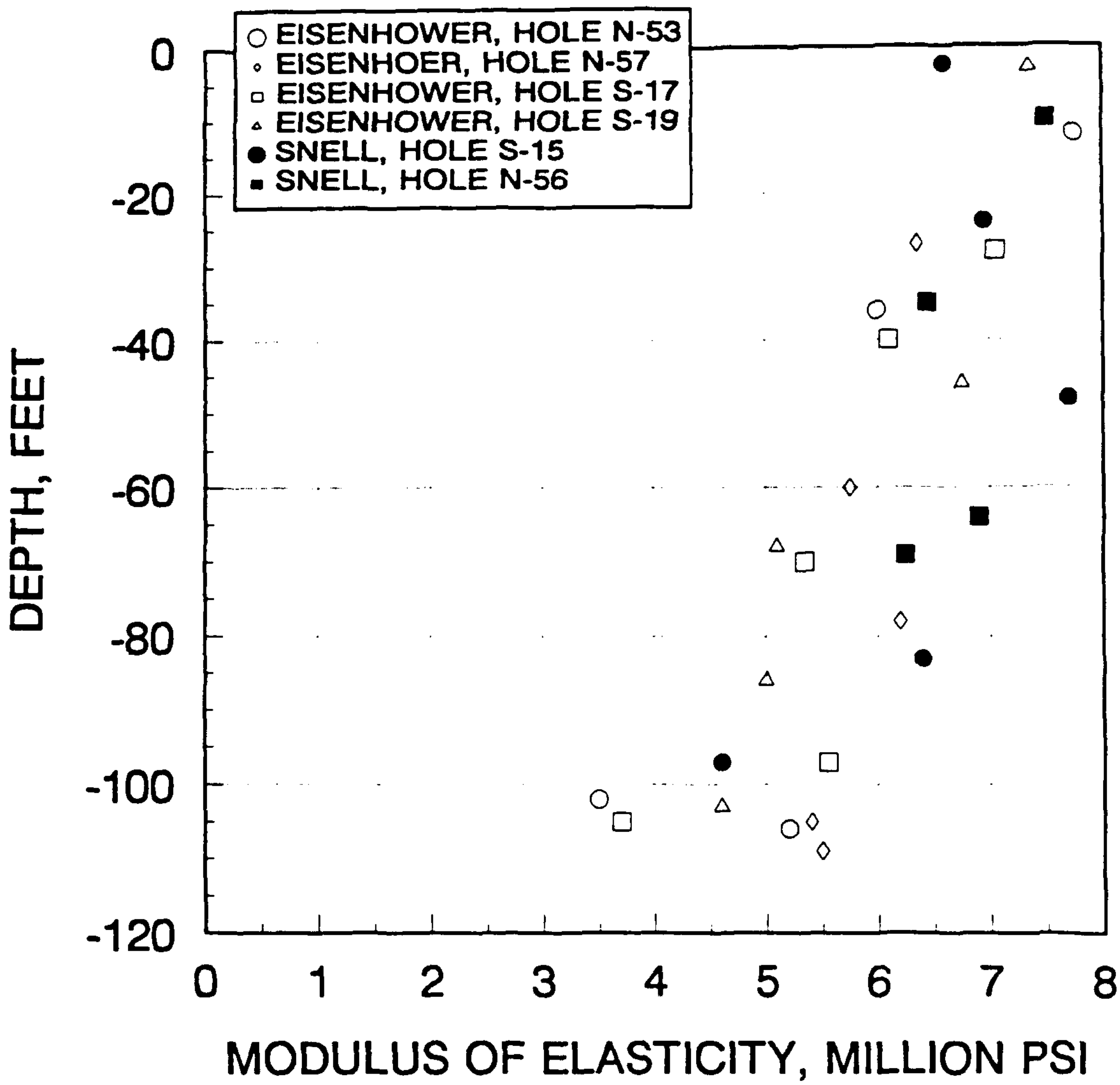


Figure 10. Variation of modulus of elasticity of the concrete with depth

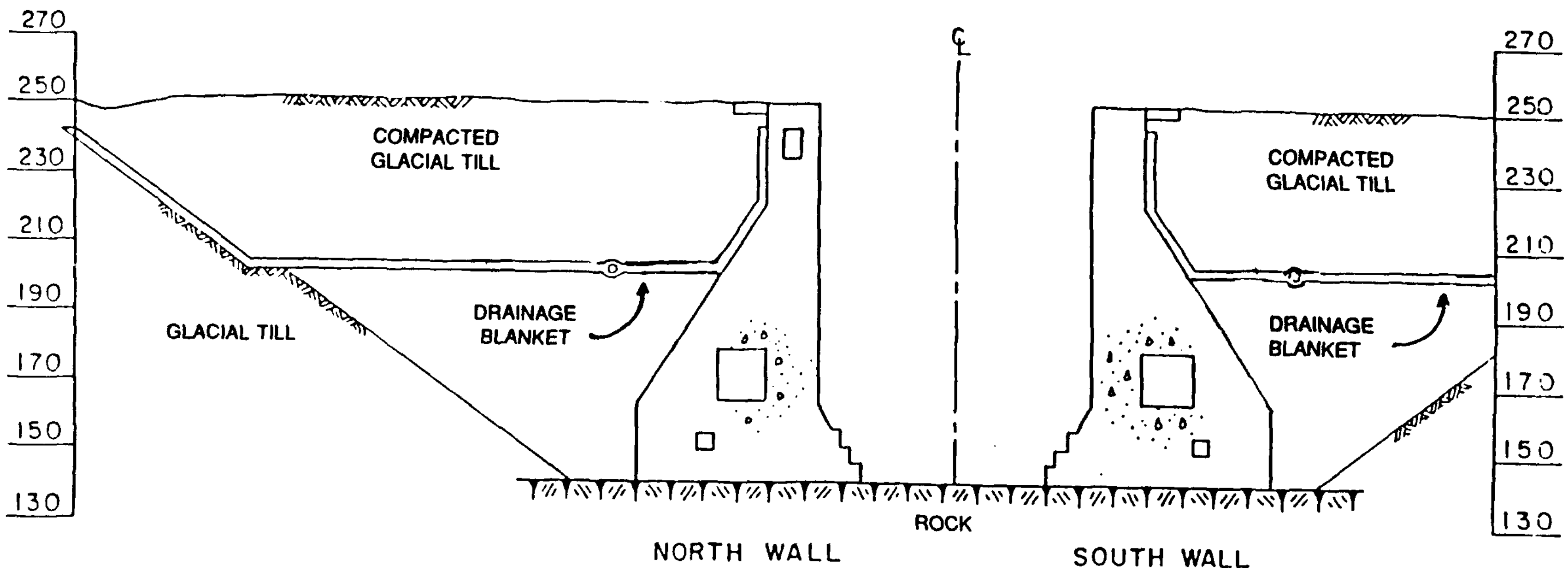


Figure 11. Typical cross section through the chamber at Eisenhower Lock

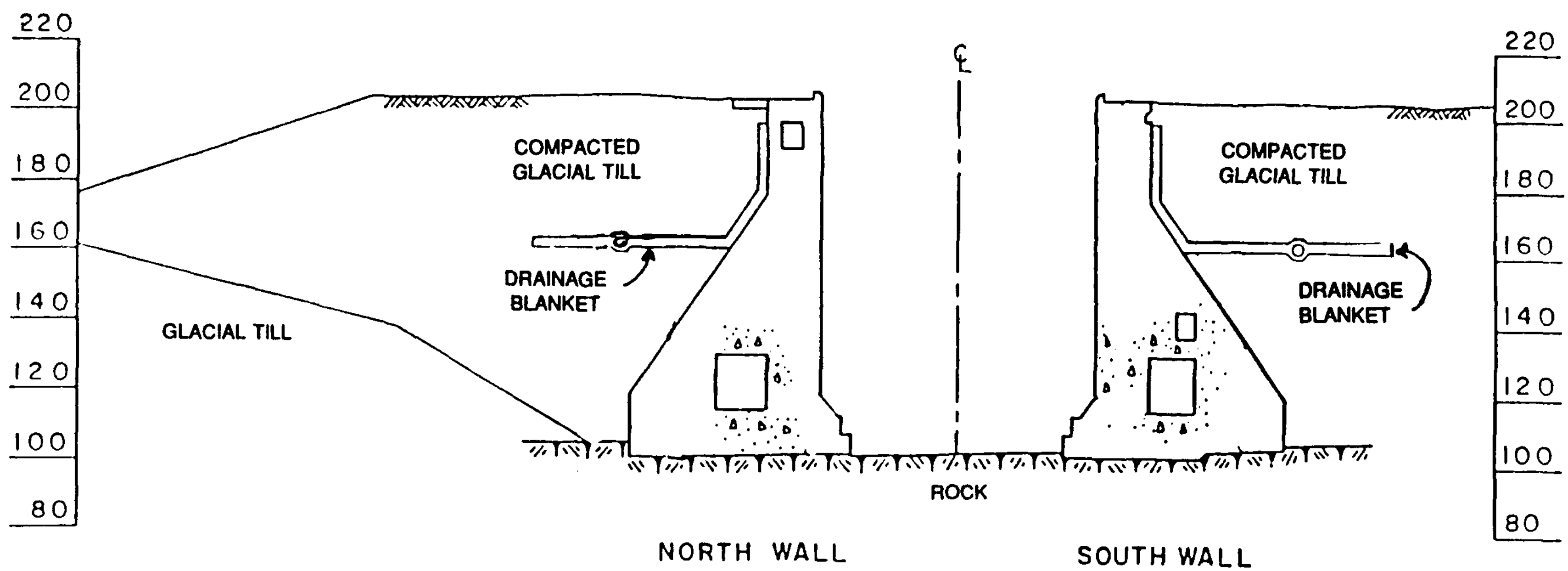


Figure 12. Typical cross section through the chamber at Snell Lock

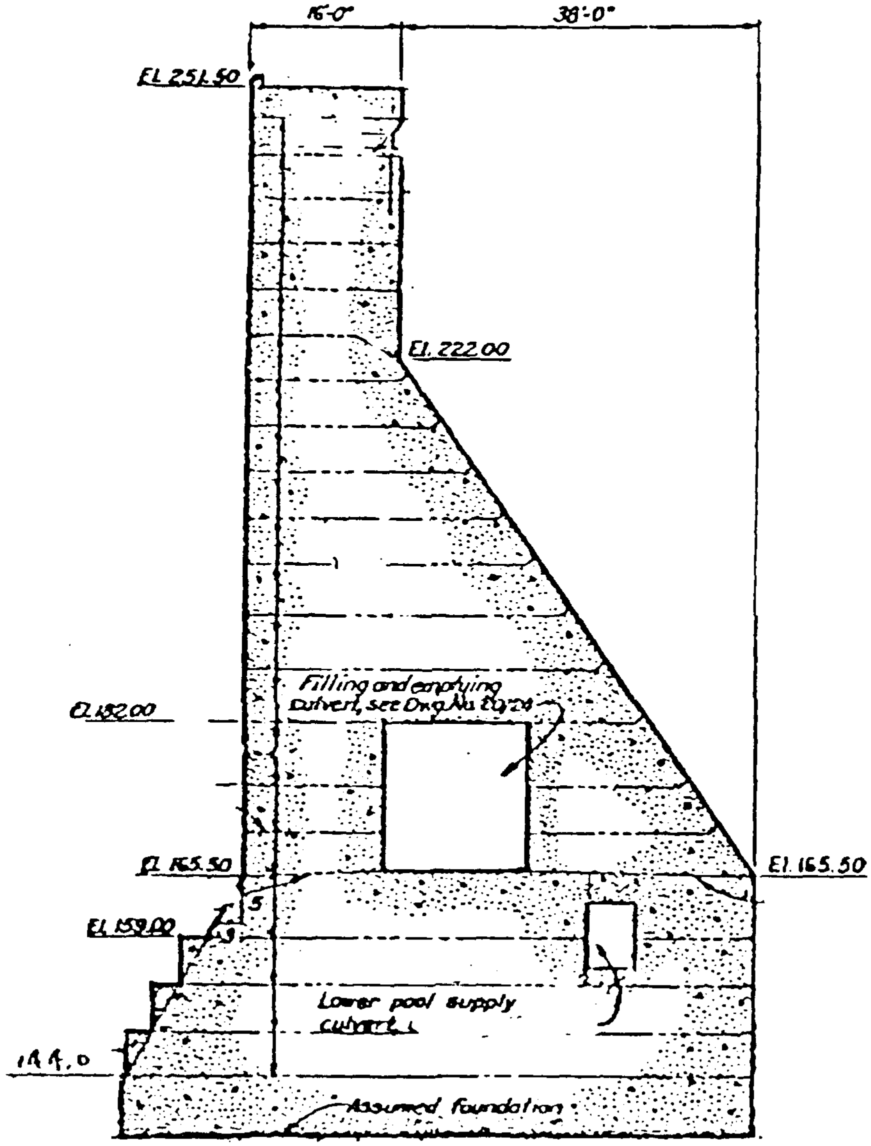


Figure 13. Typical cross section of chamber wall monolith on the south side at Eisenhower Lock

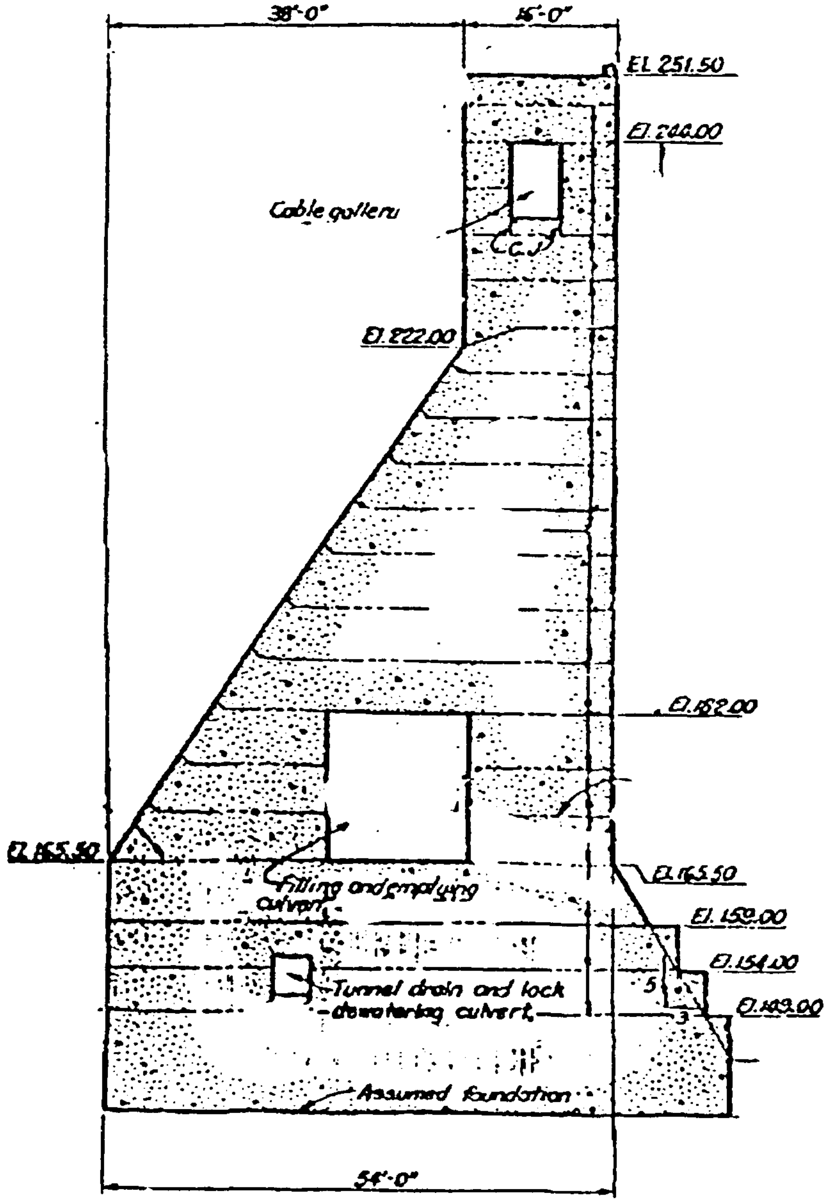


Figure 14. Typical cross section of chamber wall monolith on the north side at Eisenhower Lock

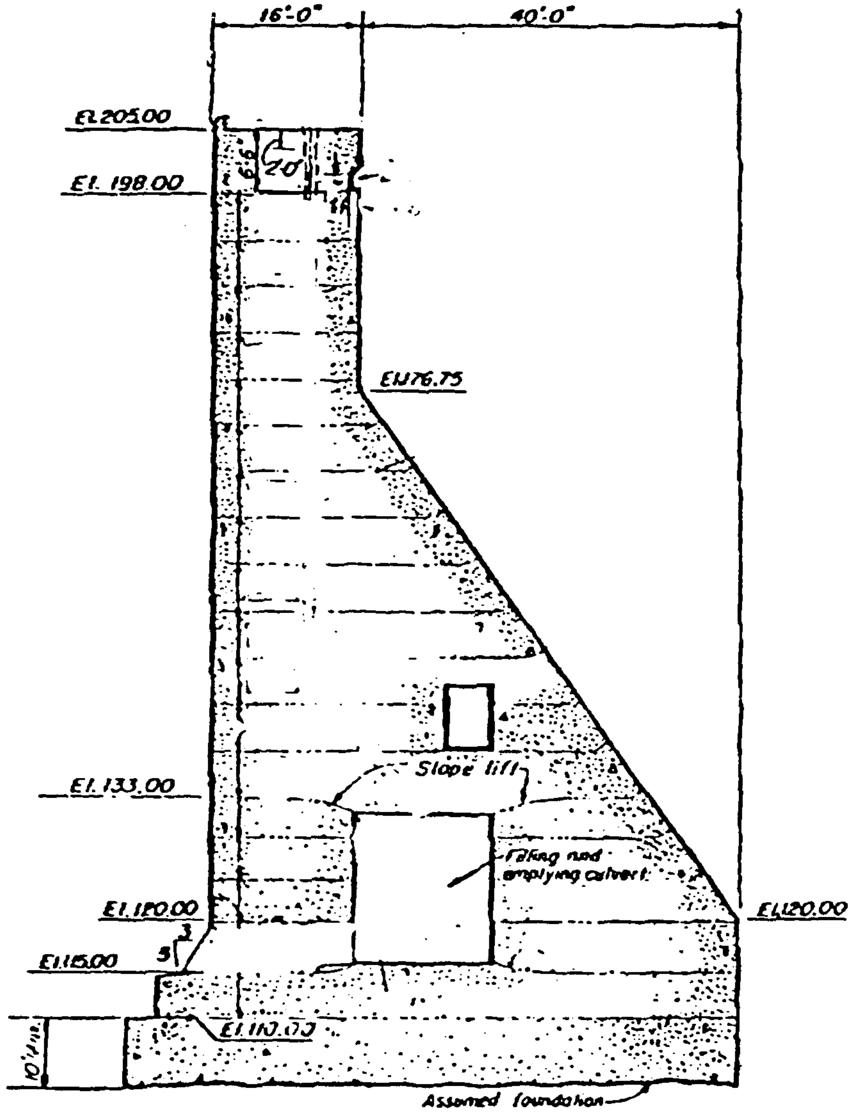


Figure 15. Typical cross section of chamber wall monolith on the south side at Snell Lock

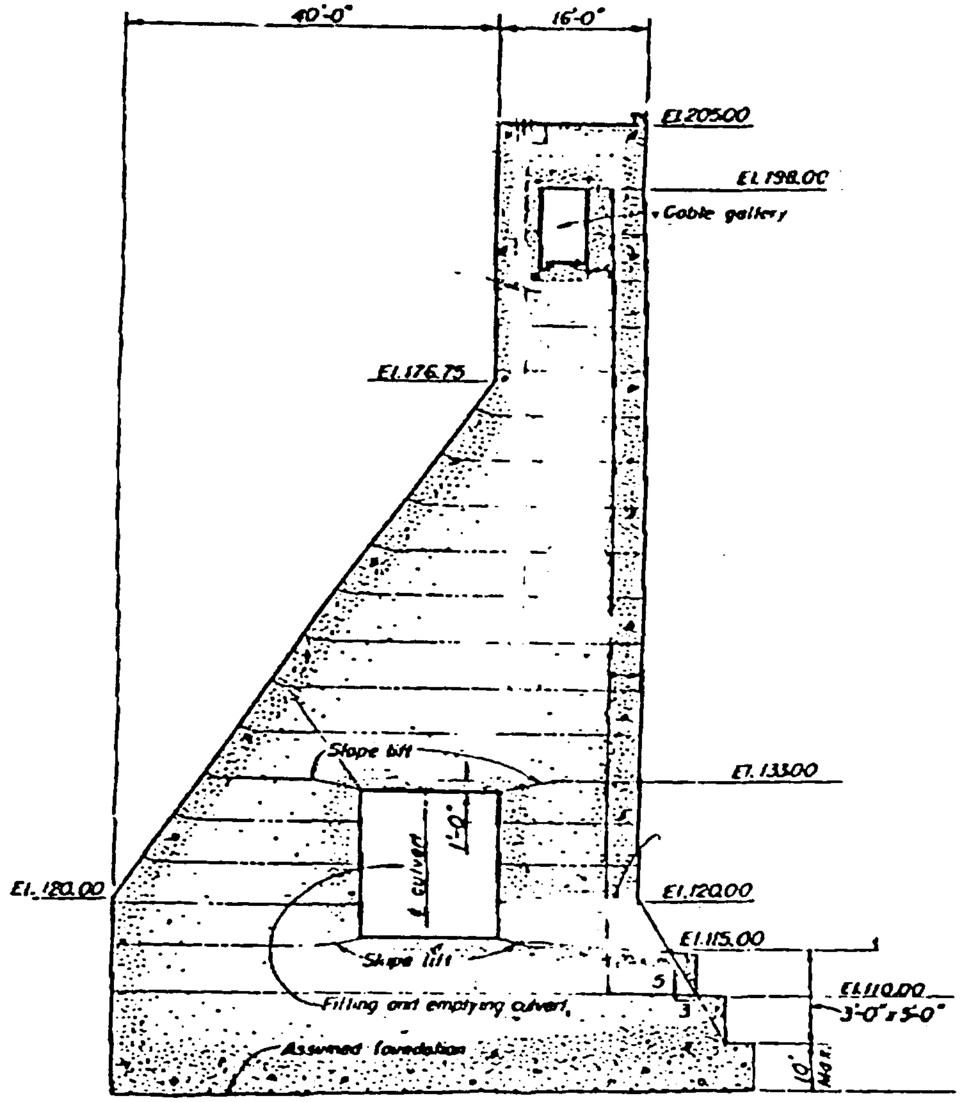


Figure 16. Typical cross section of chamber wall monolith on the north side at Snell Lock

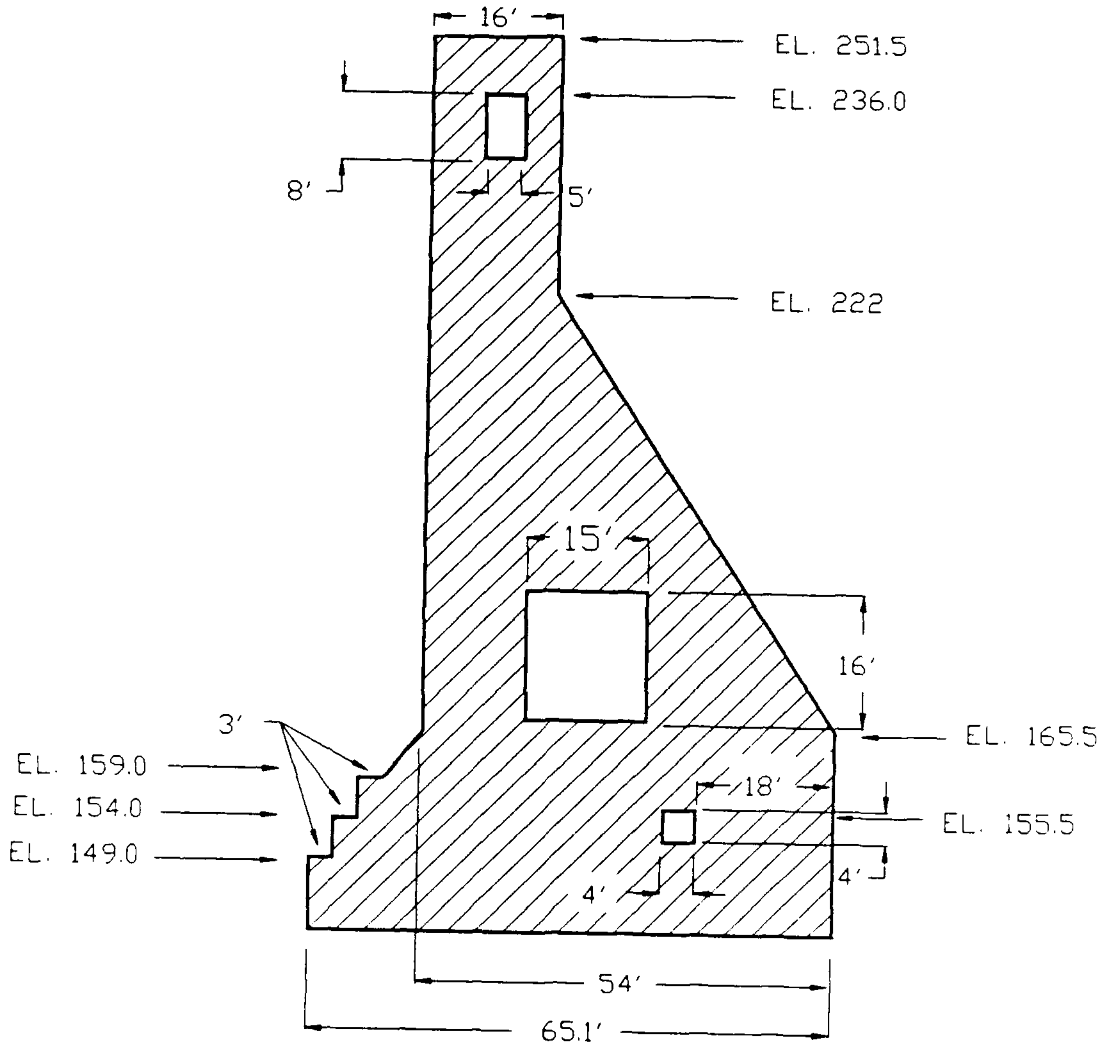


Figure 17. Basic elevations and dimensions for the model of north chamber wall monoliths at Eisenhower Lock

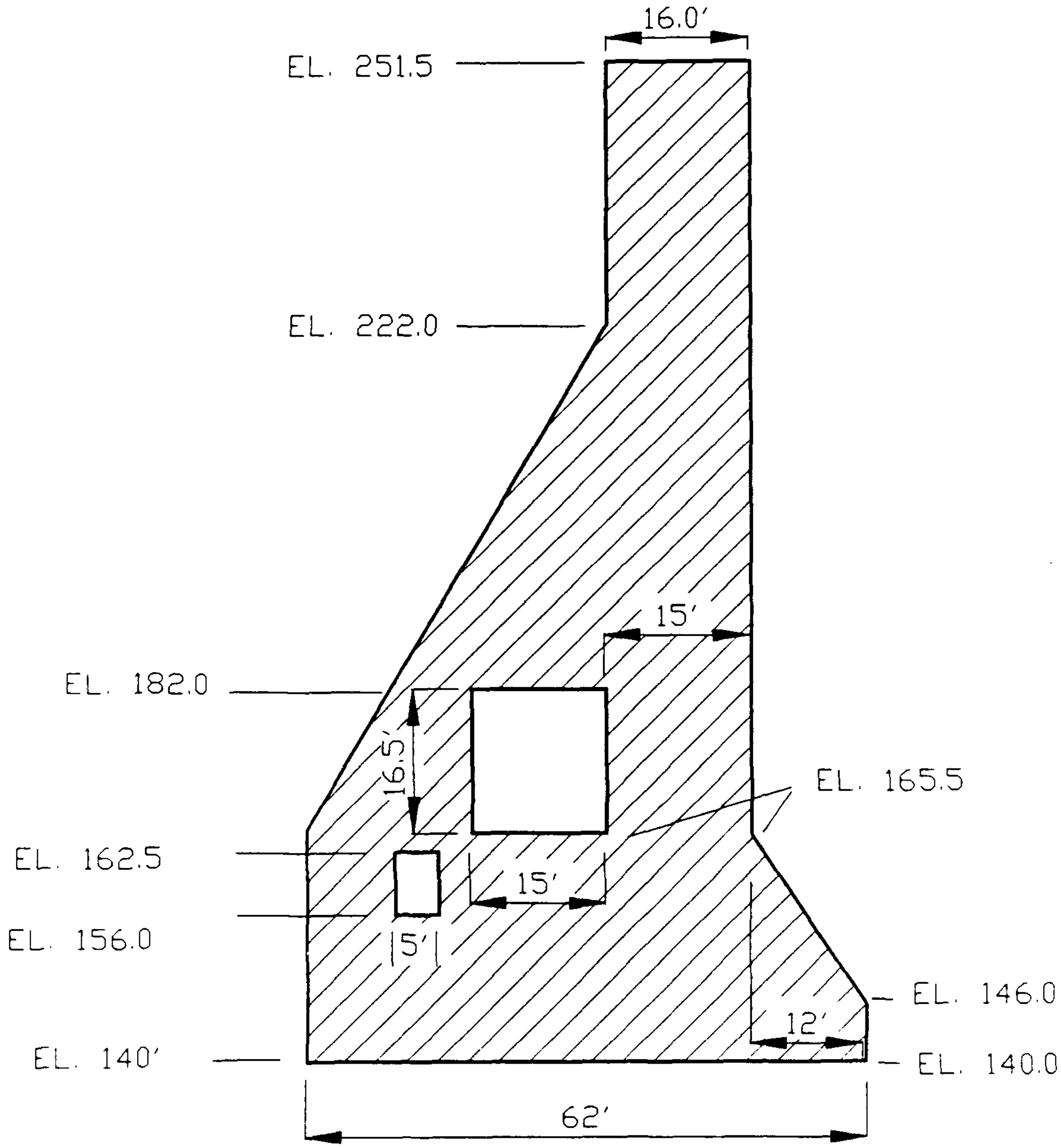


Figure 18. Basic elevations and dimensions for the model of south chamber wall monoliths at Eisenhower Lock

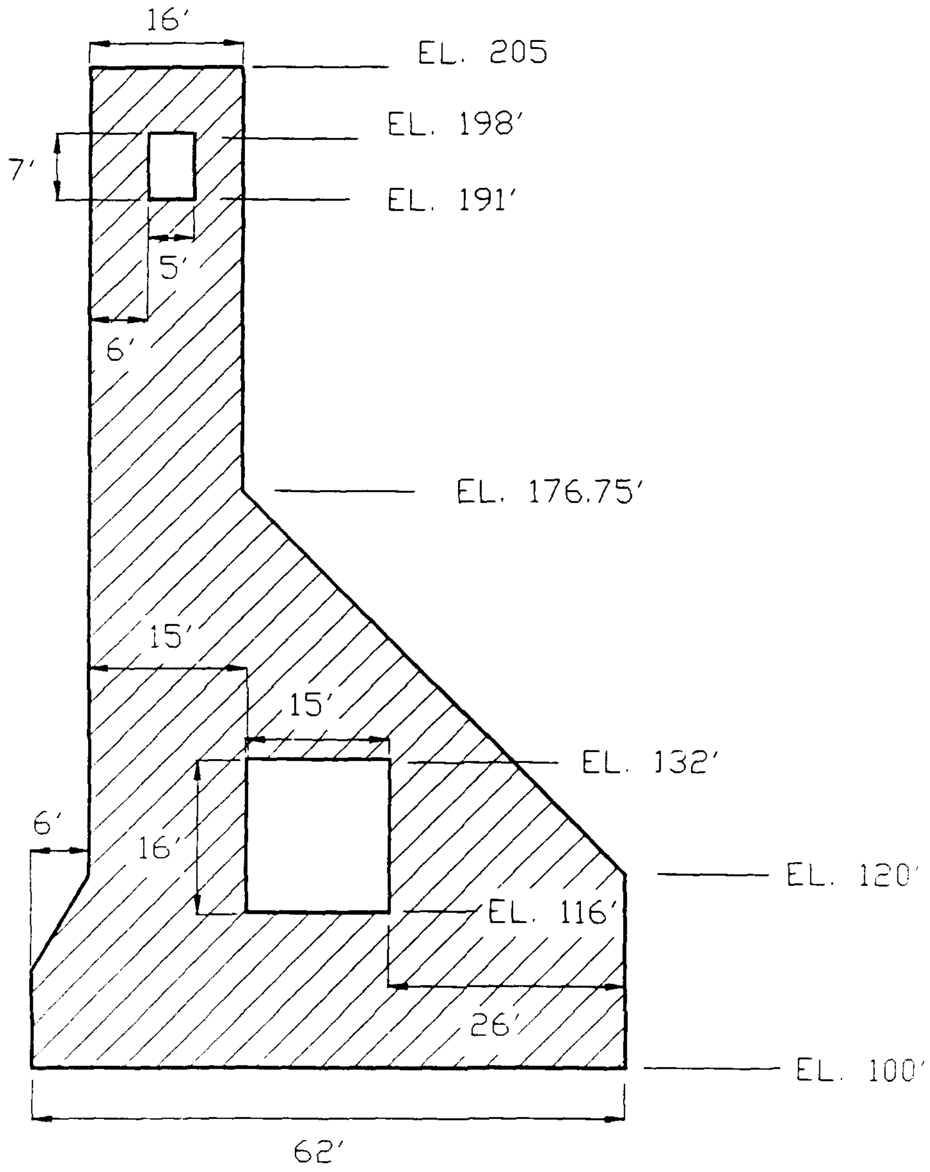


Figure 19. Basic elevations and dimensions for the model of north chamber wall monoliths at Snell Lock

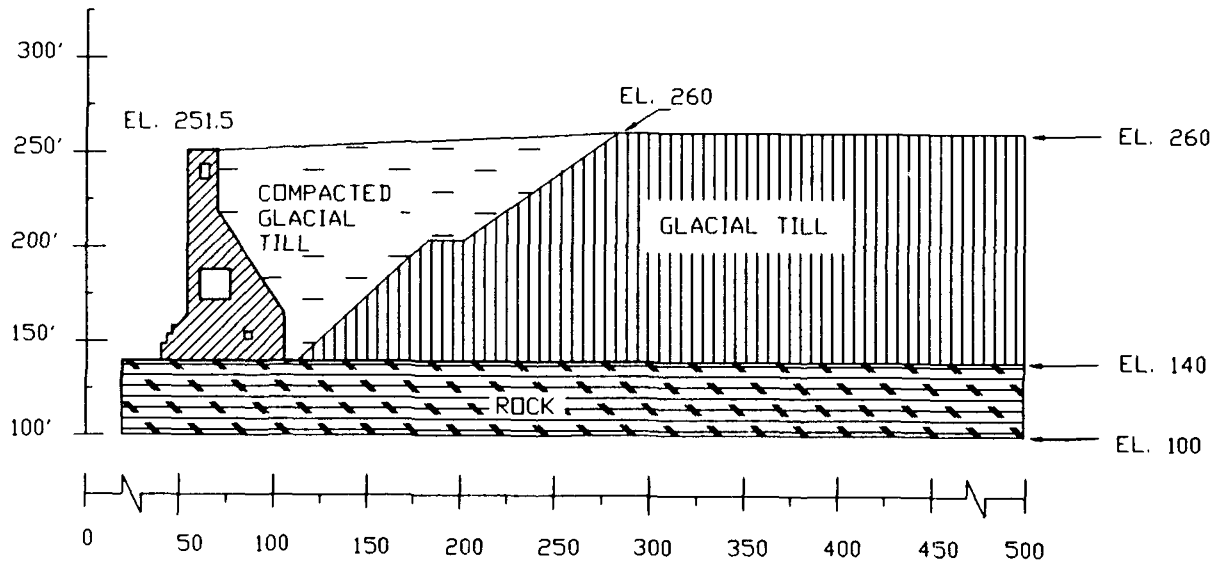


Figure 20. Configuration for the model of north chamber wall monoliths at Eisenhower Lock

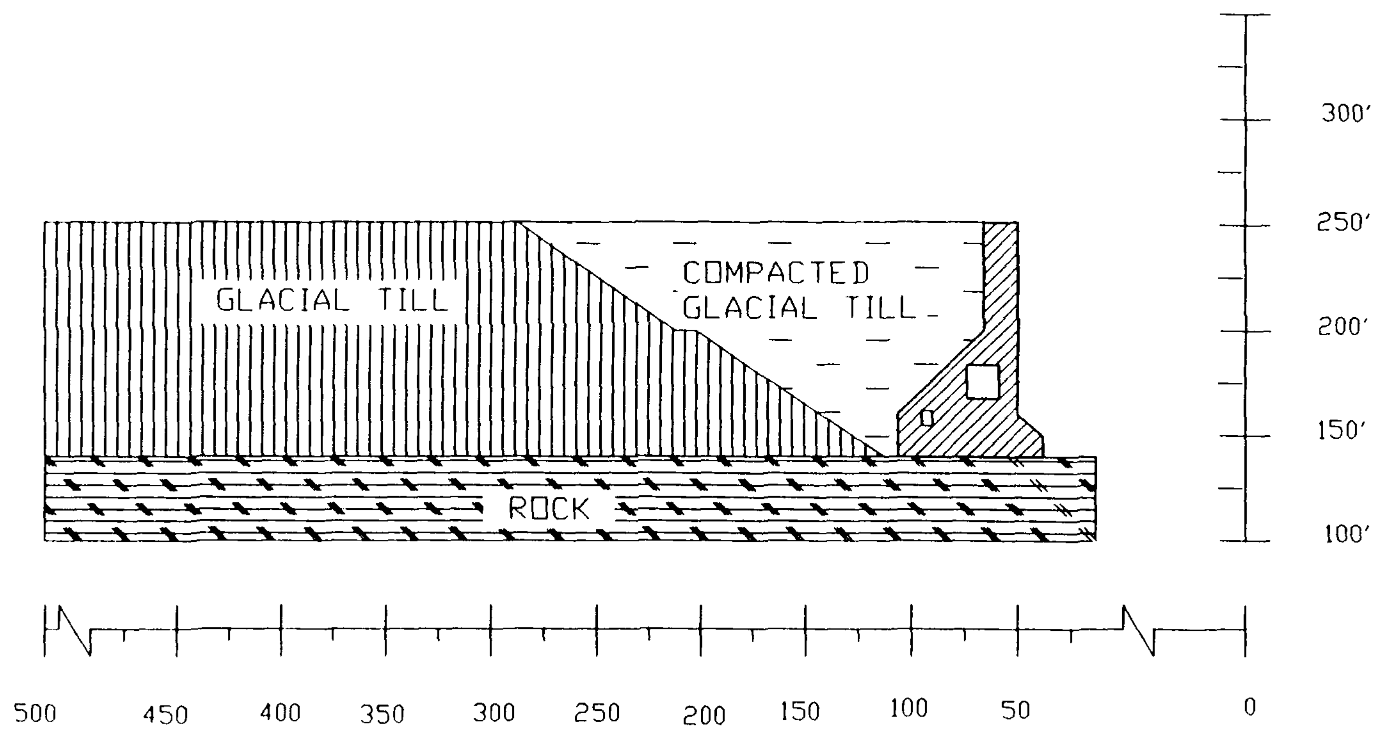


Figure 21. Configuration for the model of south chamber wall monoliths at Eisenhower Lock

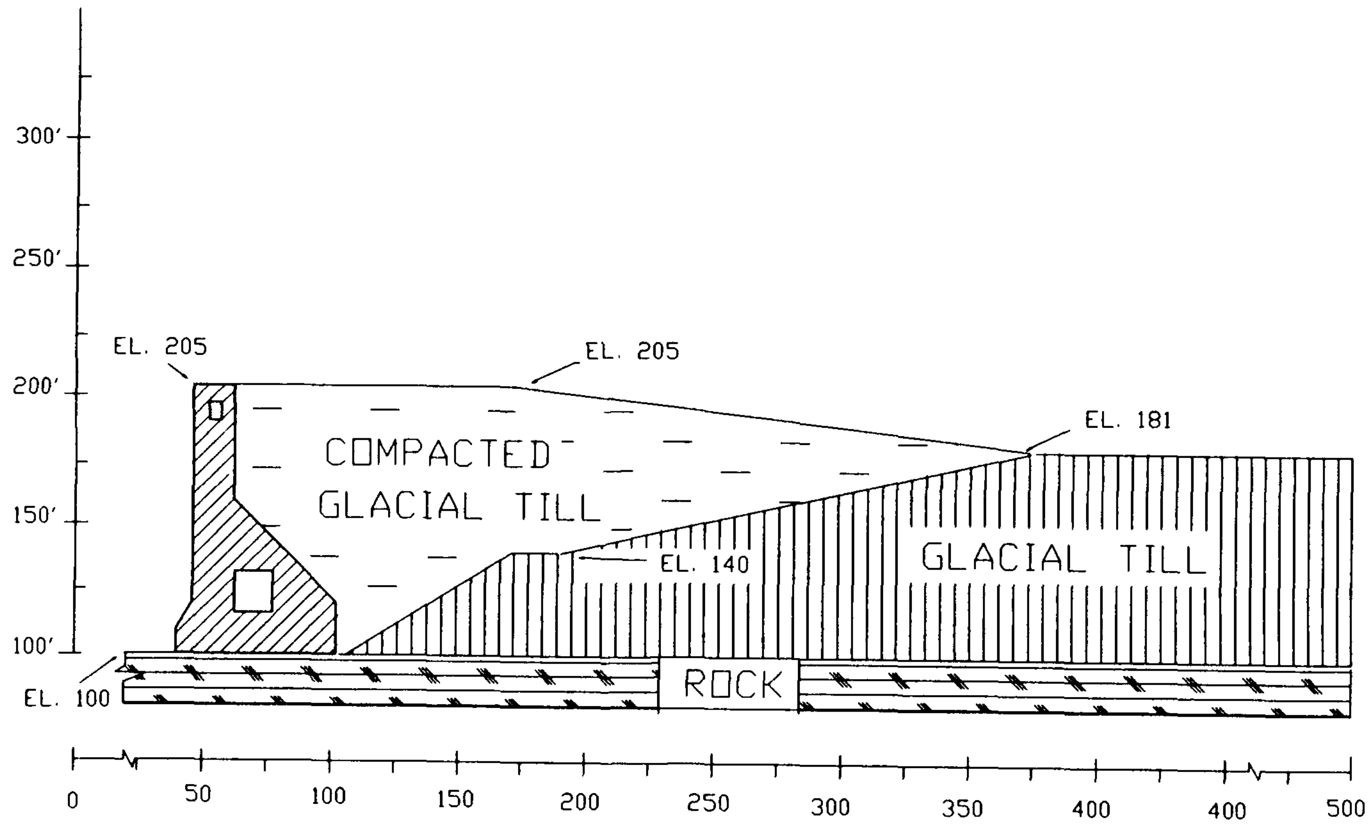


Figure 22. Configuration for the model of north chamber wall monoliths at Snell Lock

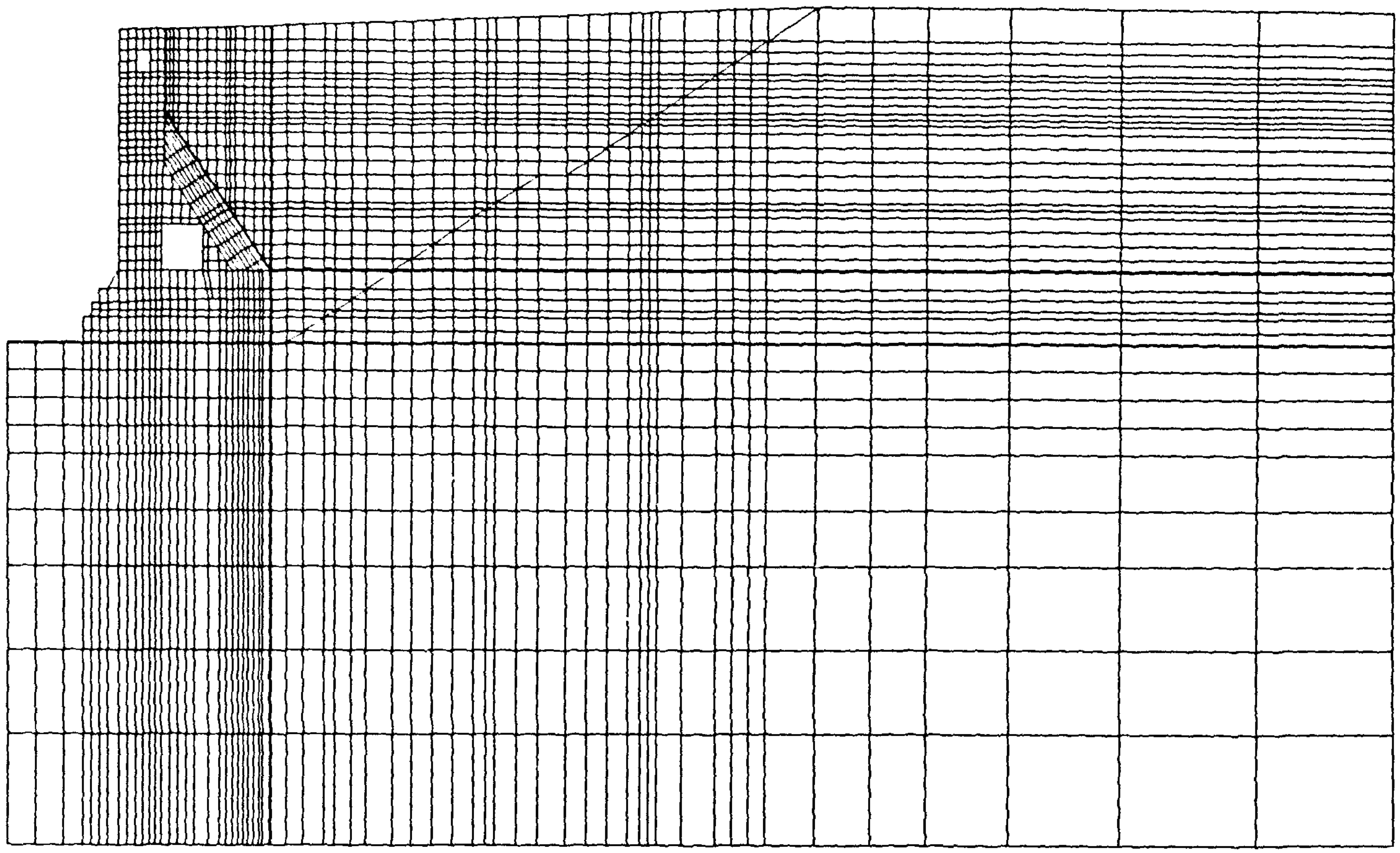


Figure 23. Finite element mesh for the model of north chamber wall monoliths at Eisenhower Lock

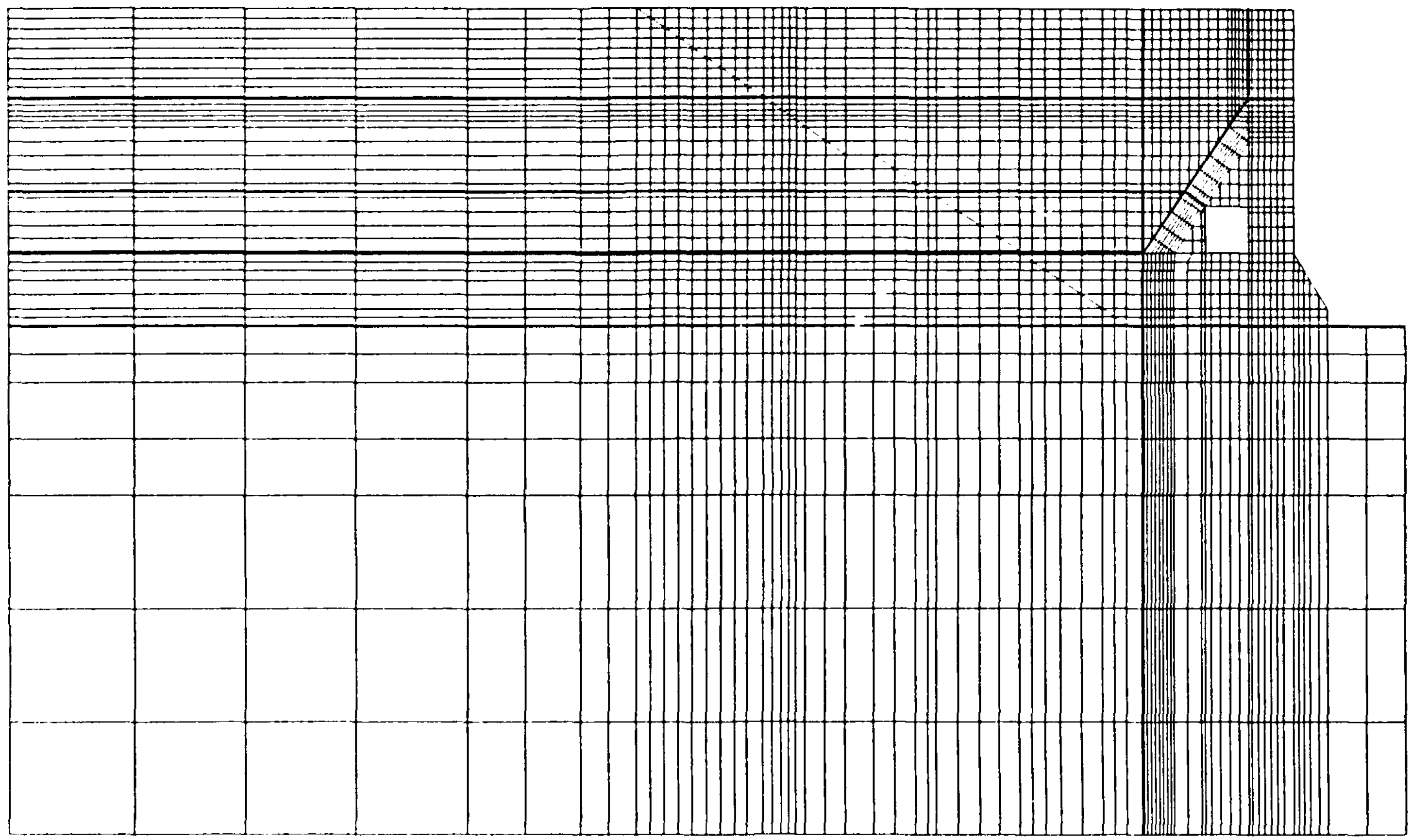


Figure 24. Finite element mesh for the model of south chamber wall monoliths at Eisenhower Lock

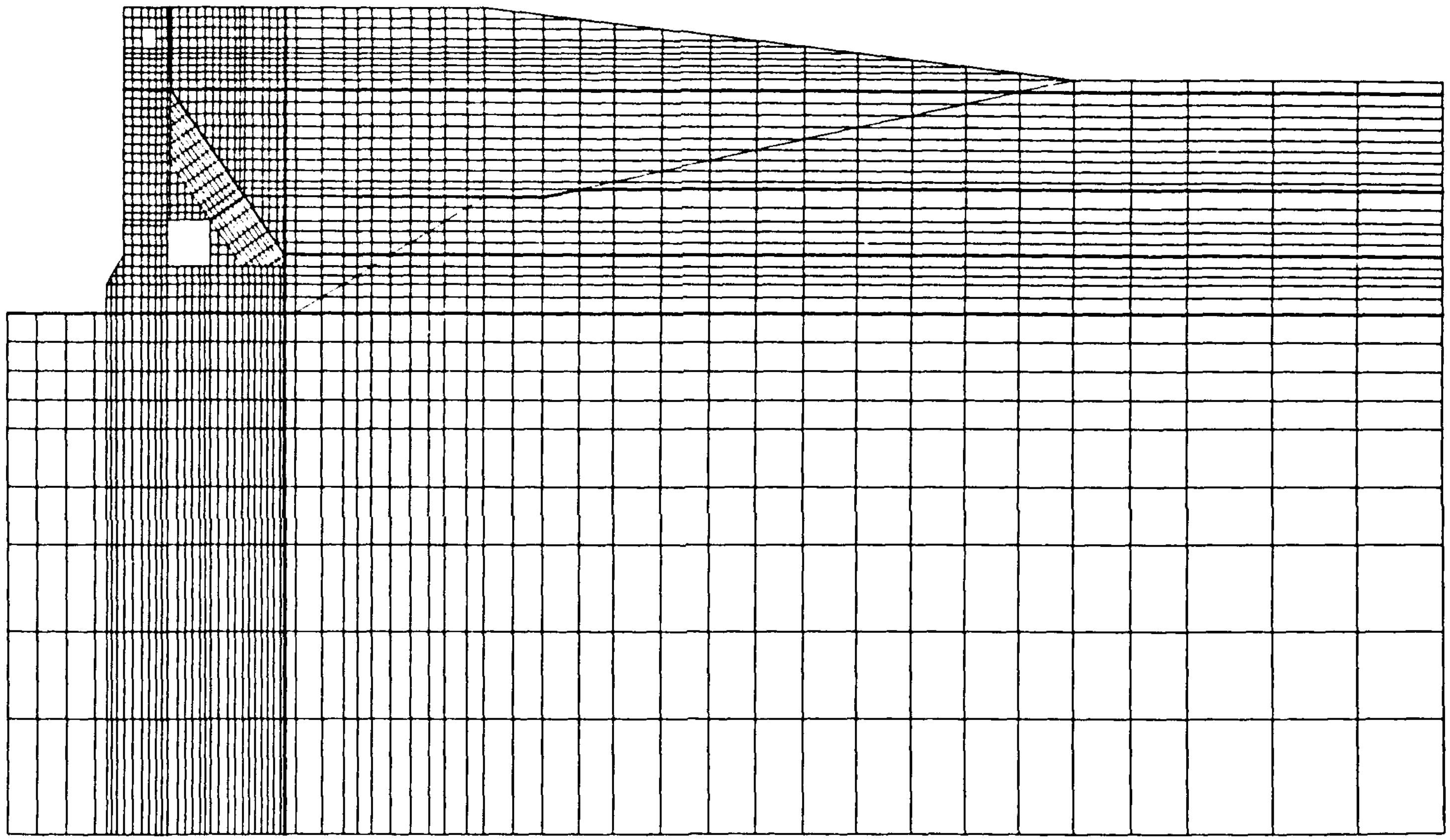


Figure 25. Finite element mesh for the model of north chamber wall monoliths at Snell Lock

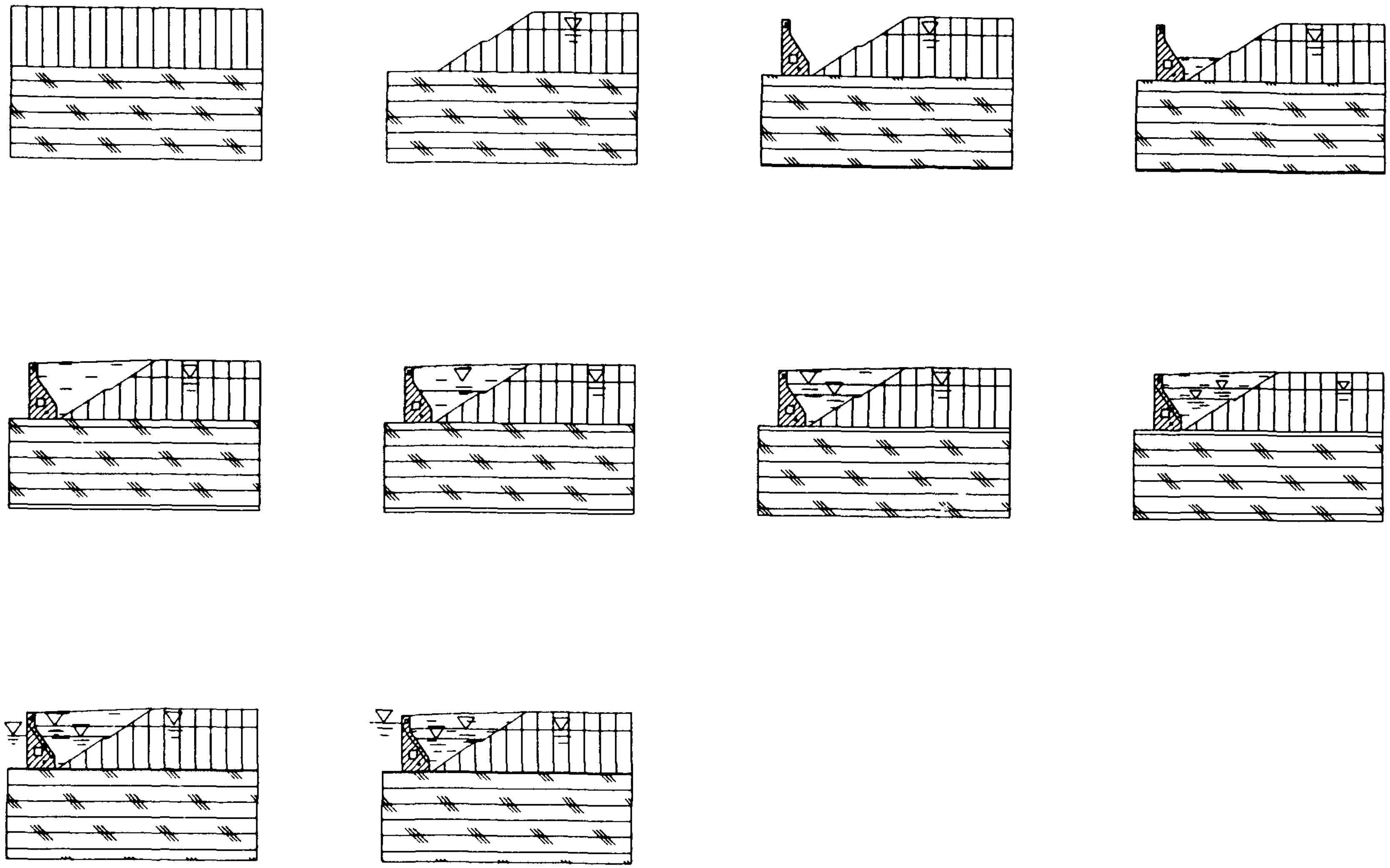


Figure 26. Schematic of the model process

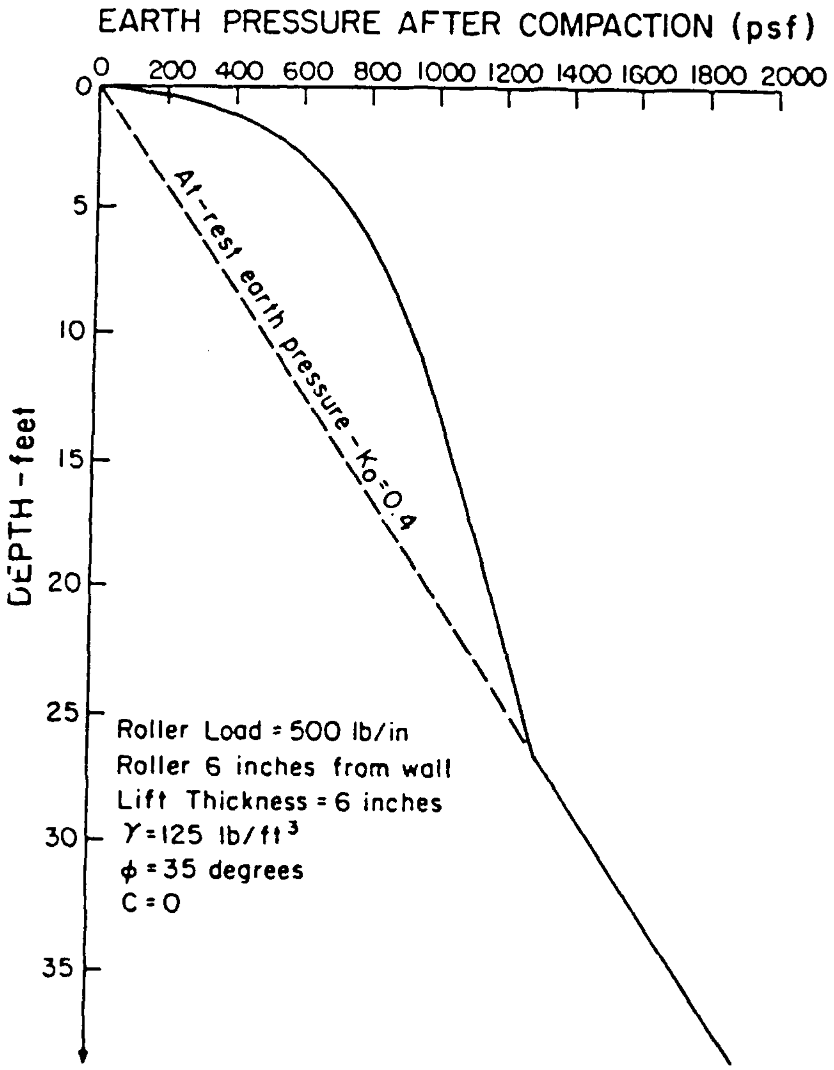
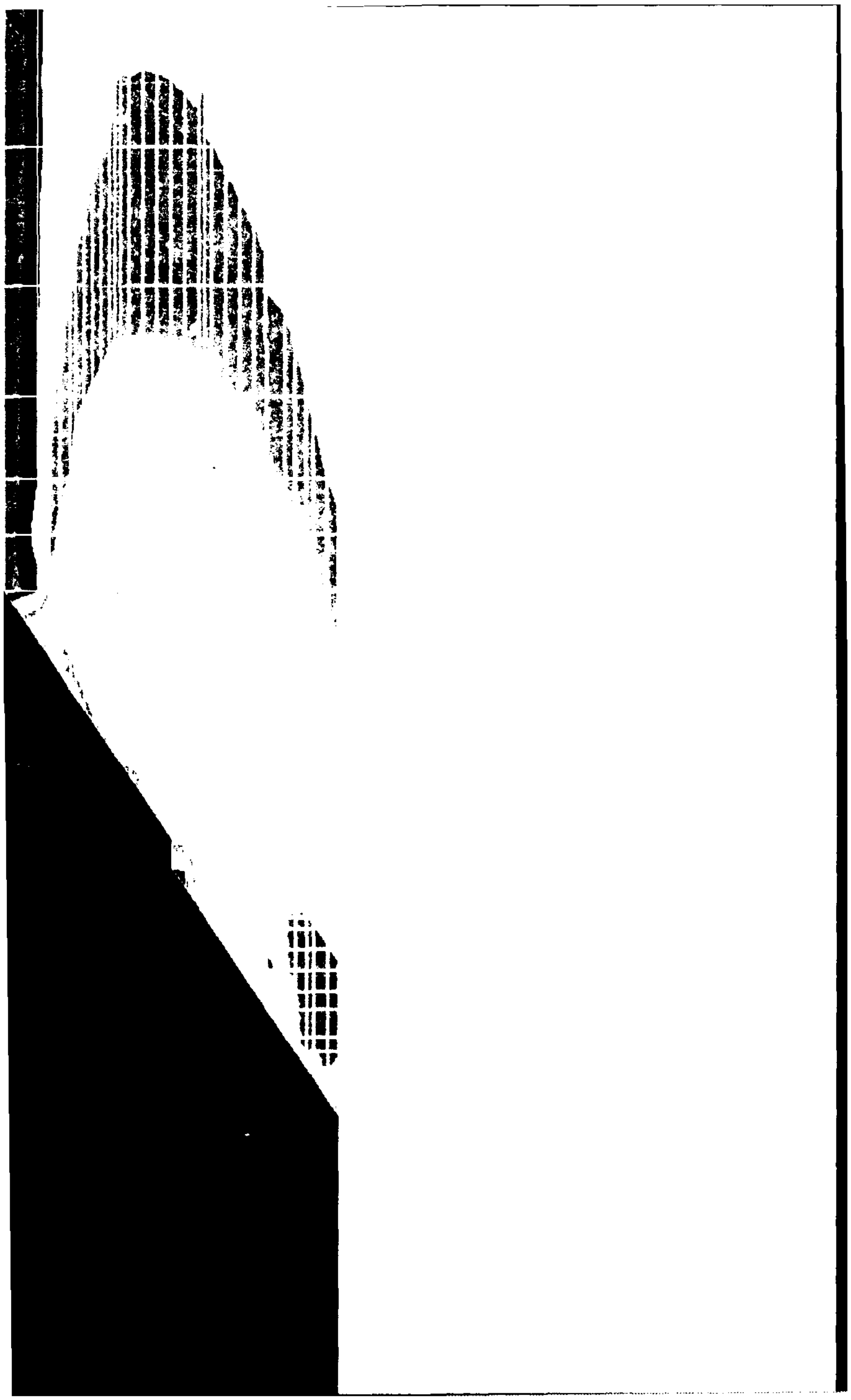
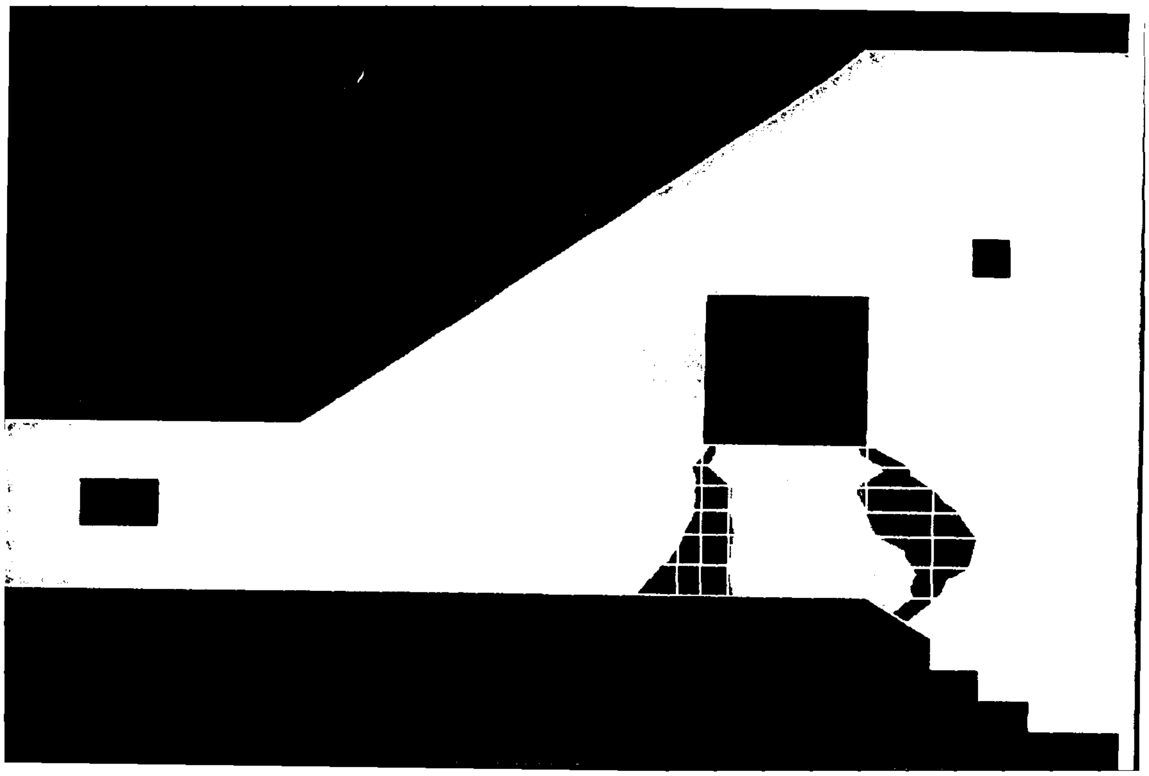
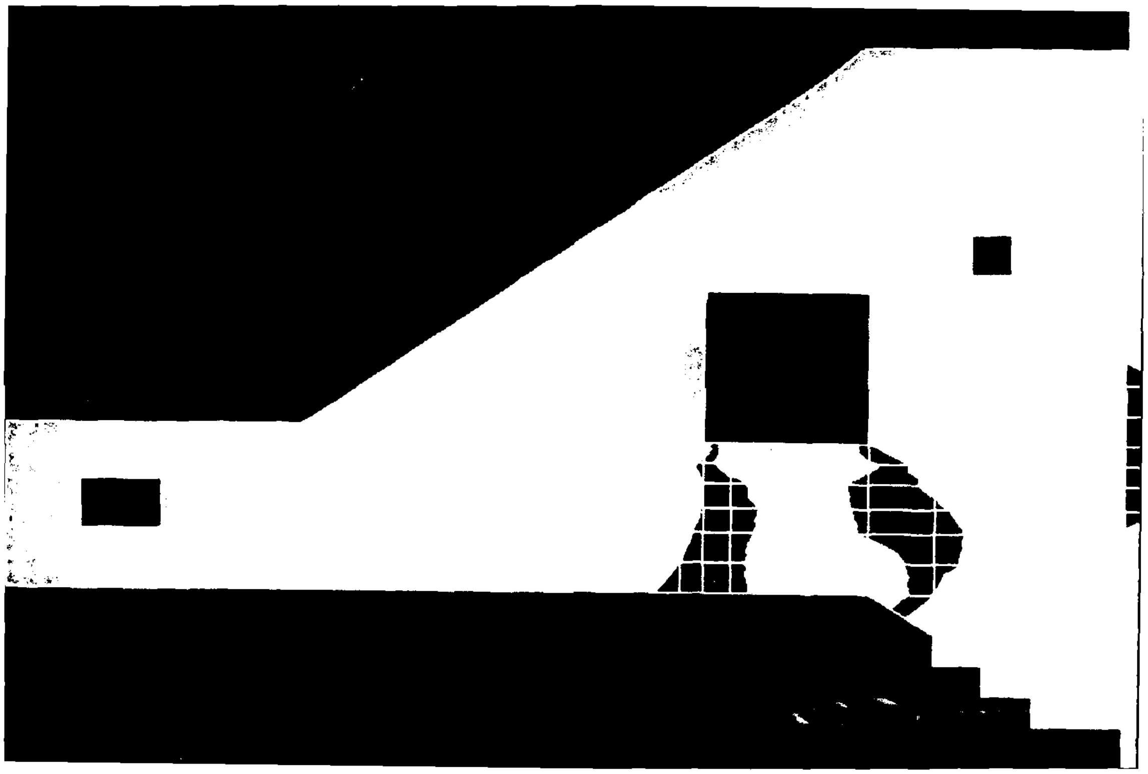
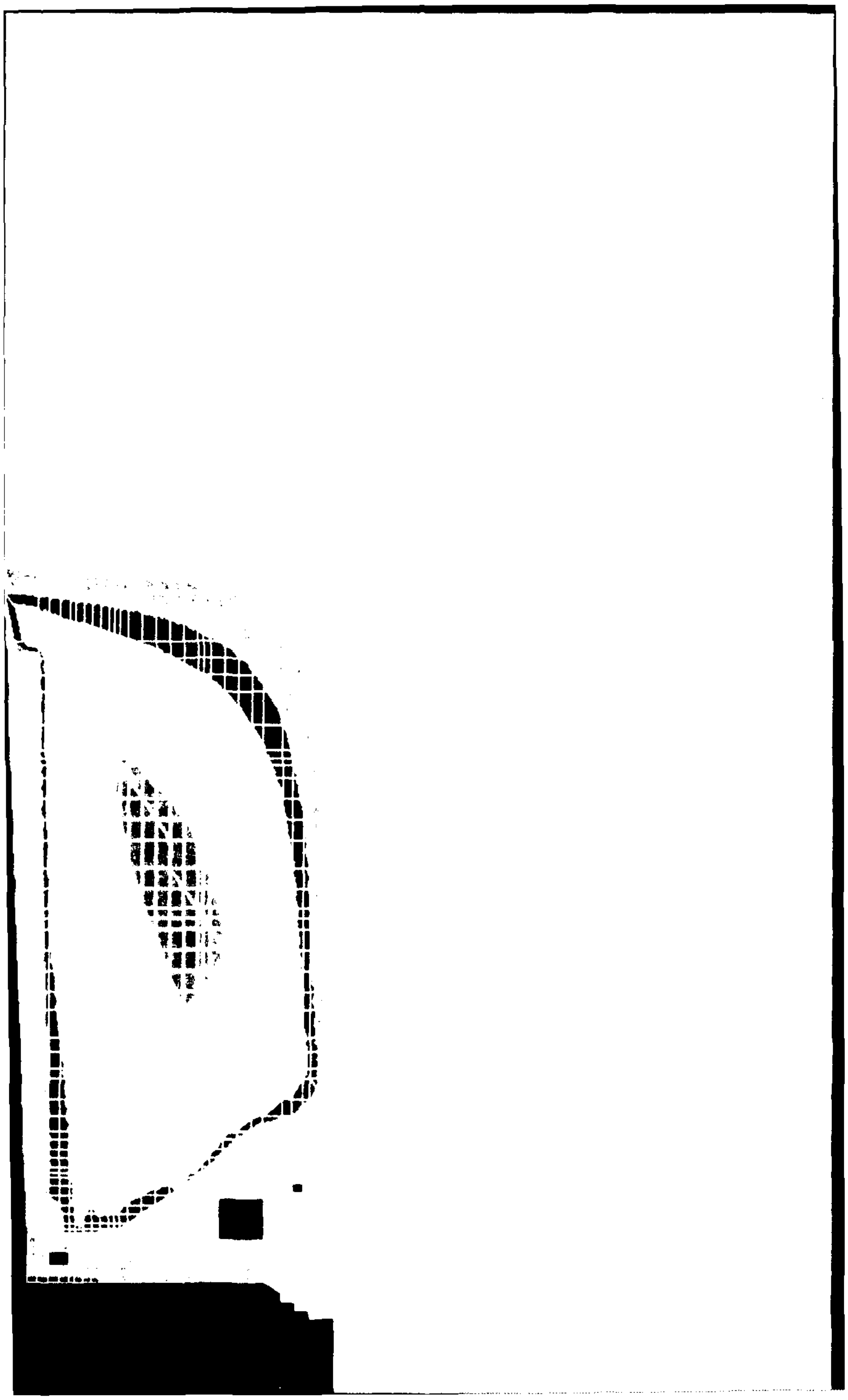


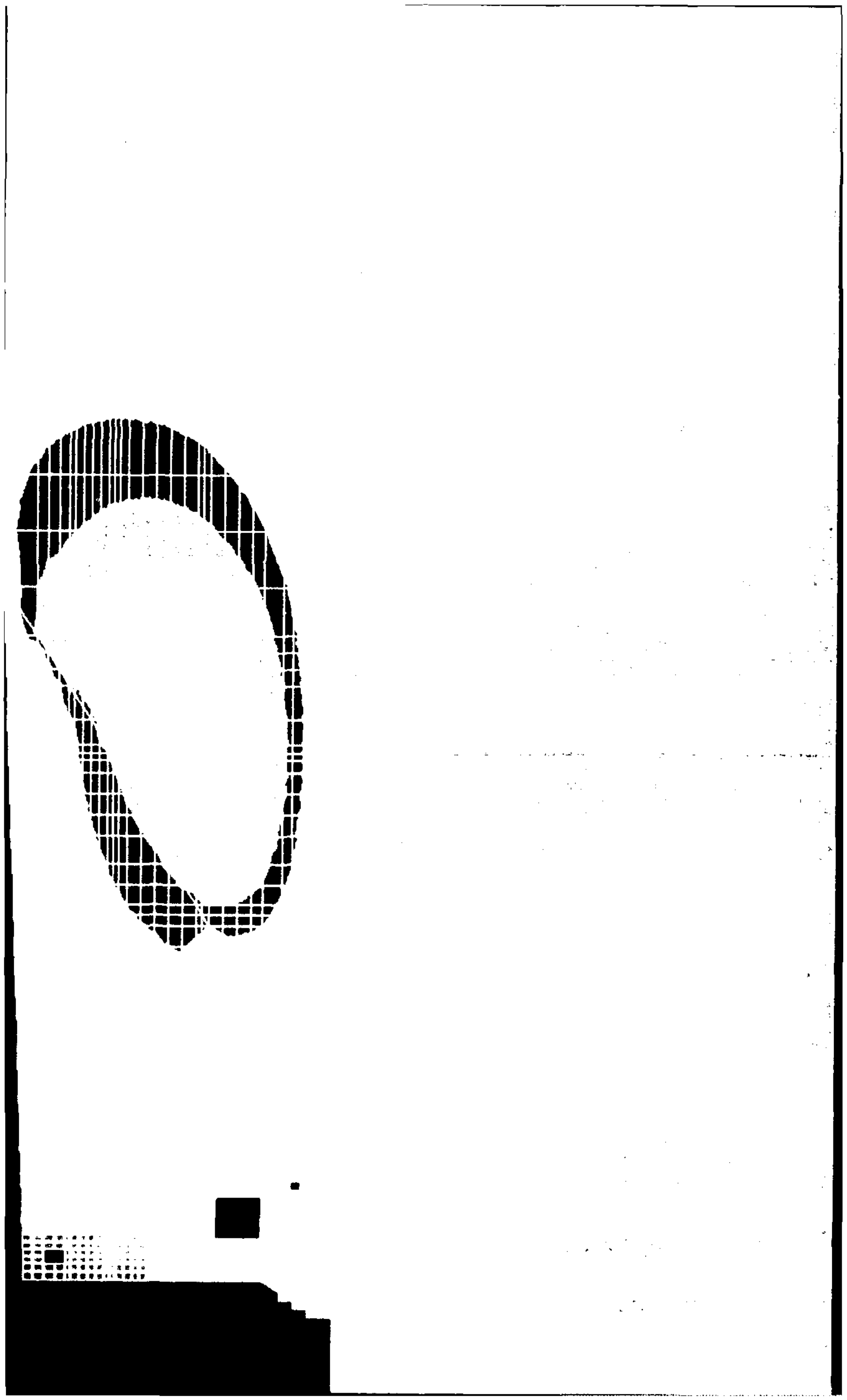
Figure 27. Effects of compaction of lateral earth pressures (Clough and Duncan 1969)











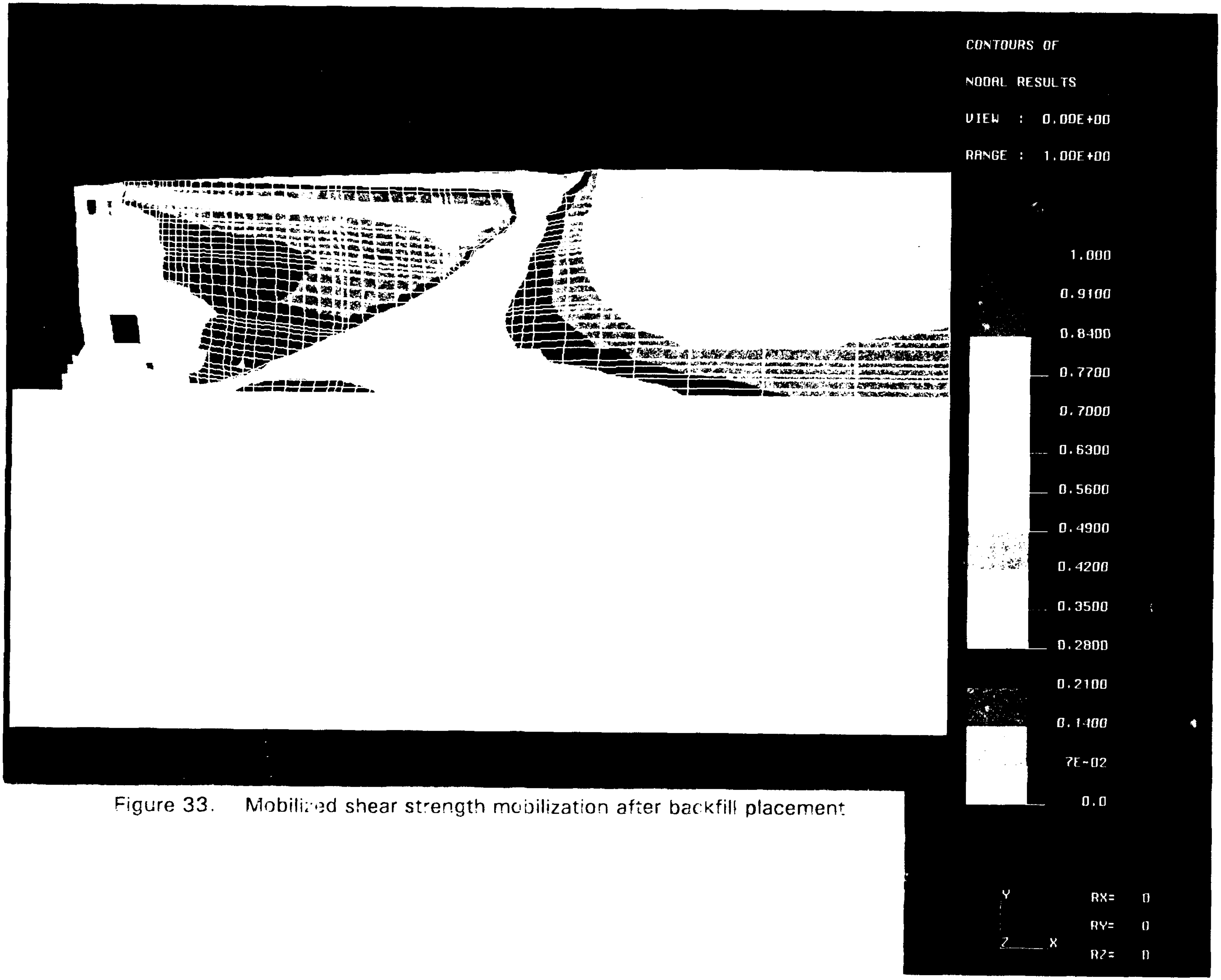


Figure 33. Mobilized shear strength mobilization after backfill placement

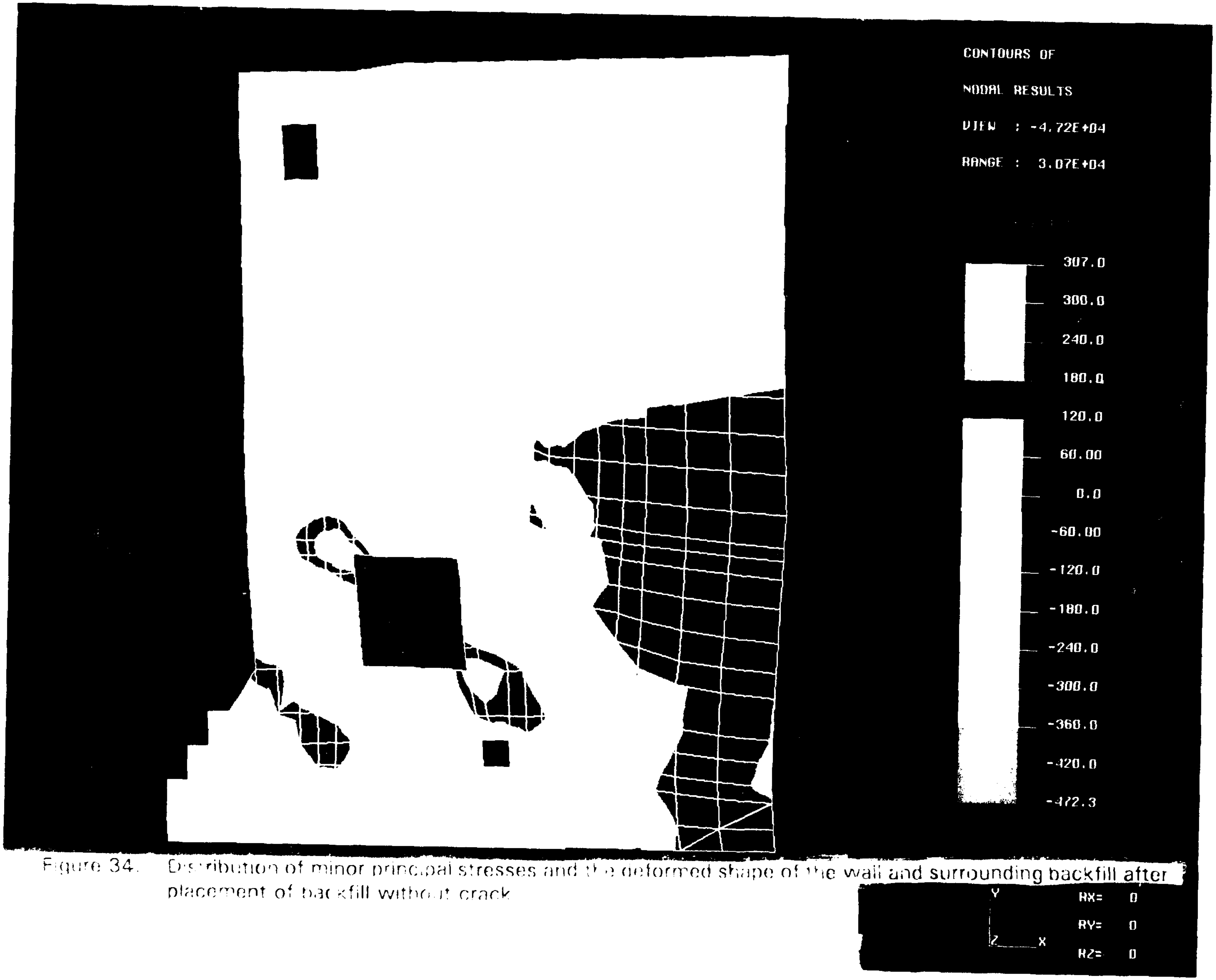


Figure 34. Distribution of minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of backfill without crack

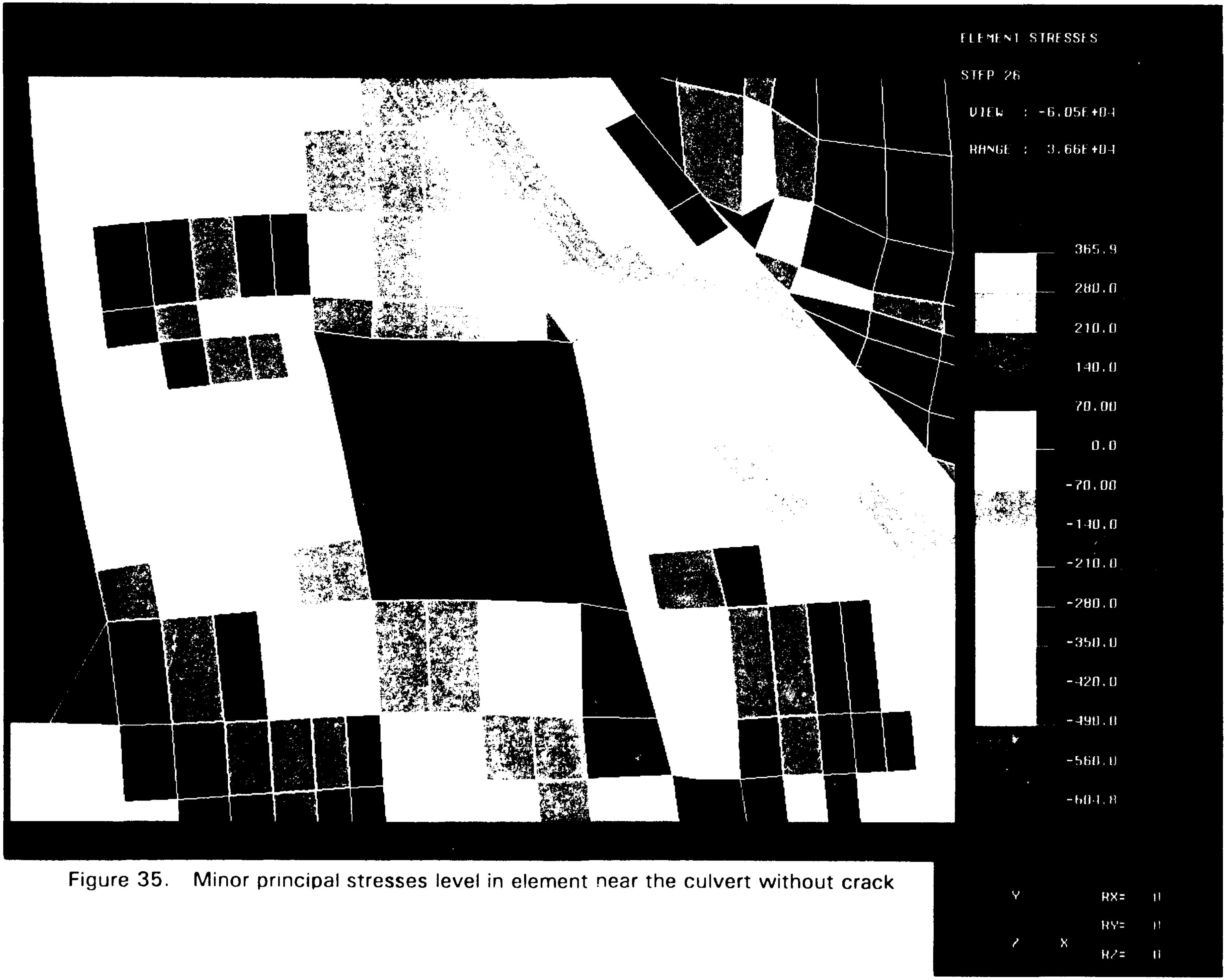


Figure 35. Minor principal stresses level in element near the culvert without crack

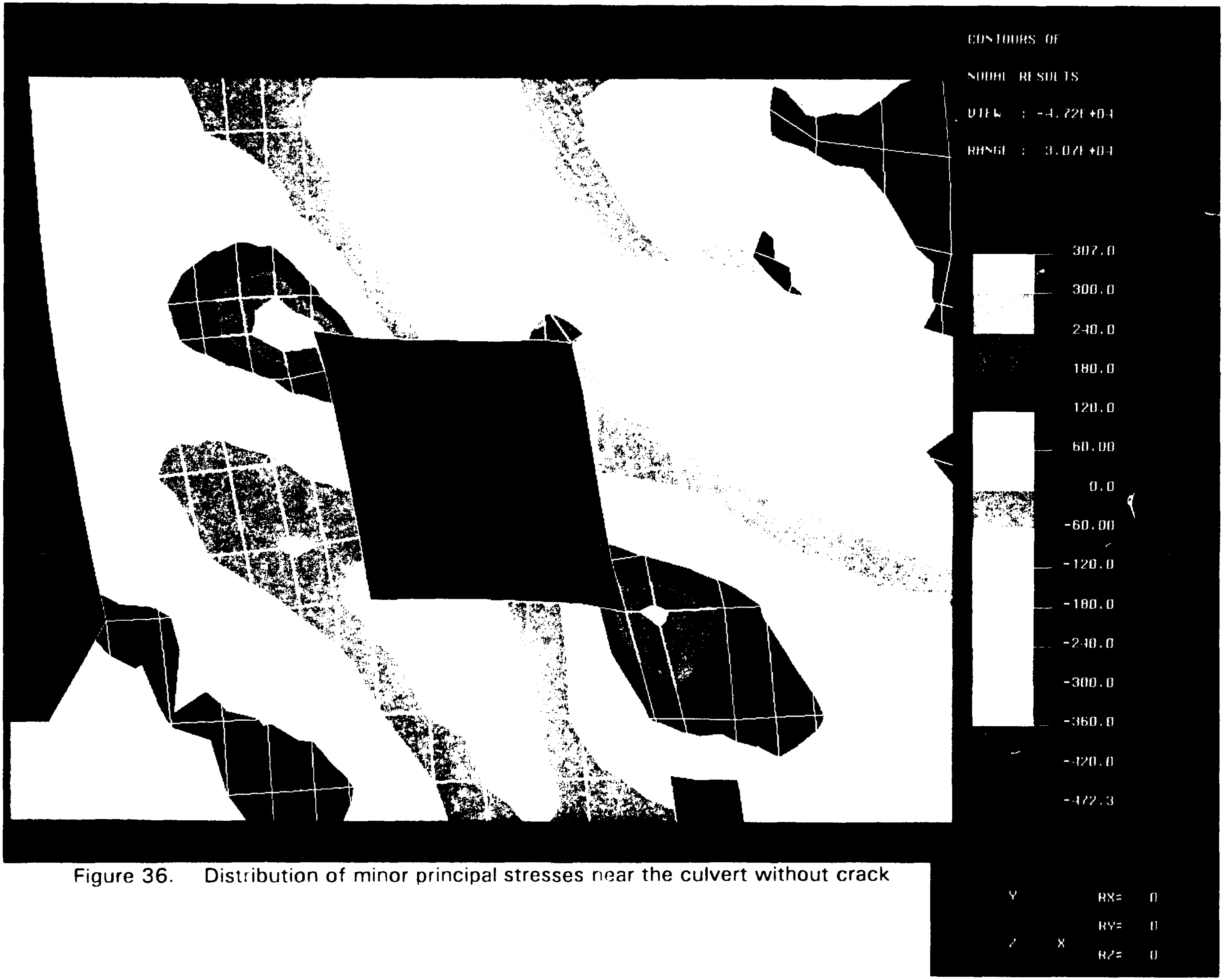


Figure 36. Distribution of minor principal stresses near the culvert without crack

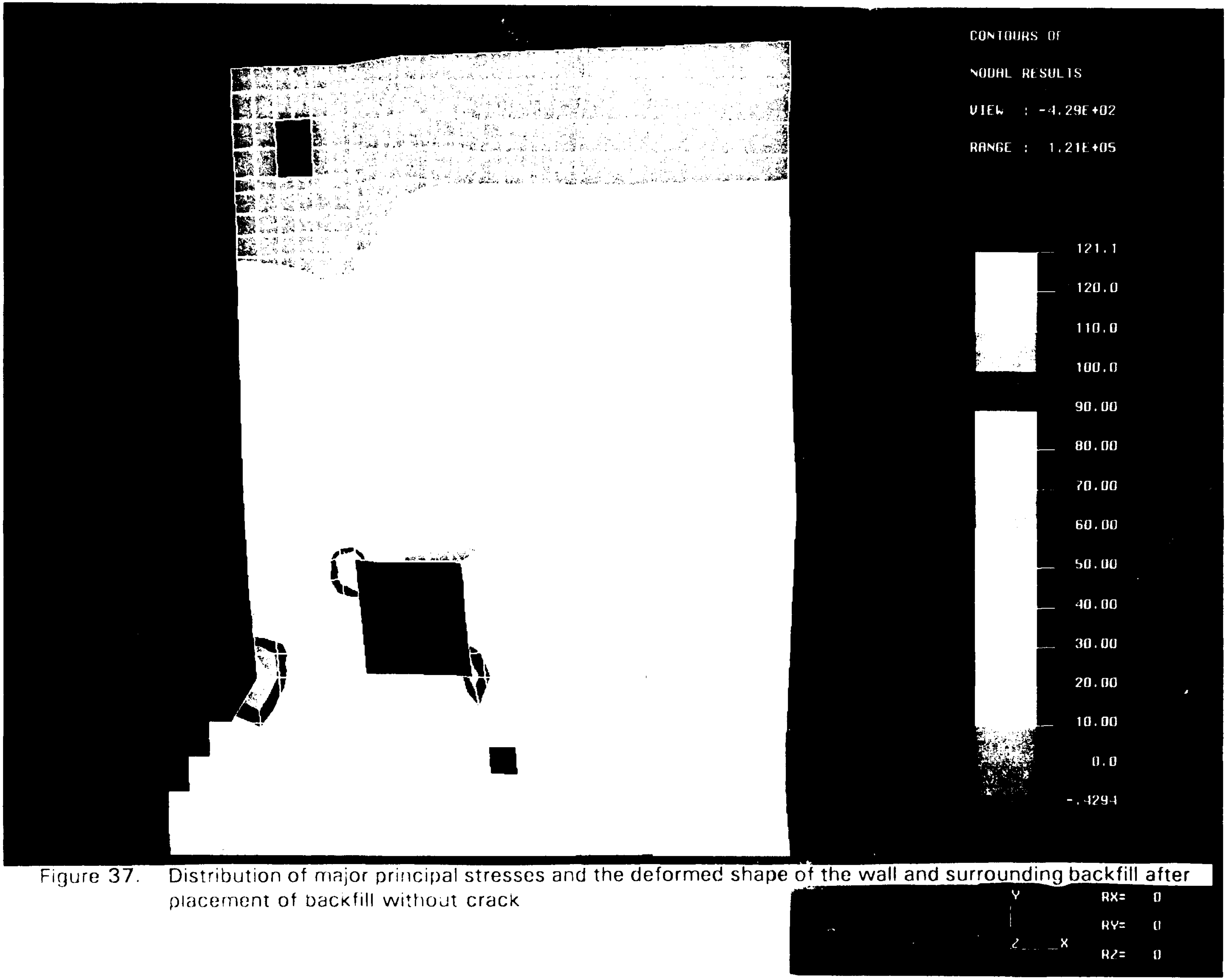


Figure 37. Distribution of major principal stresses and the deformed shape of the wall and surrounding backfill after placement of backfill without crack

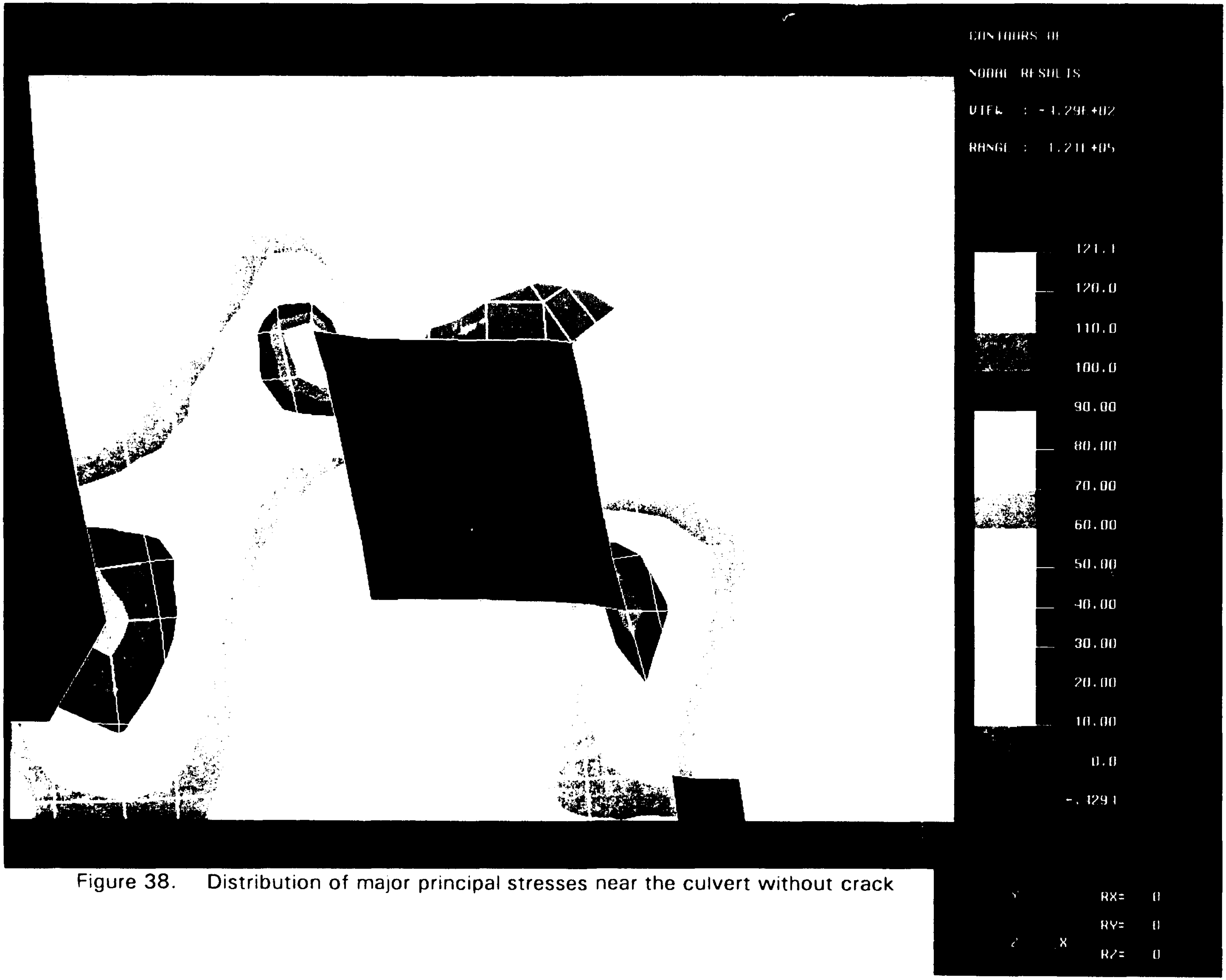


Figure 38. Distribution of major principal stresses near the culvert without crack

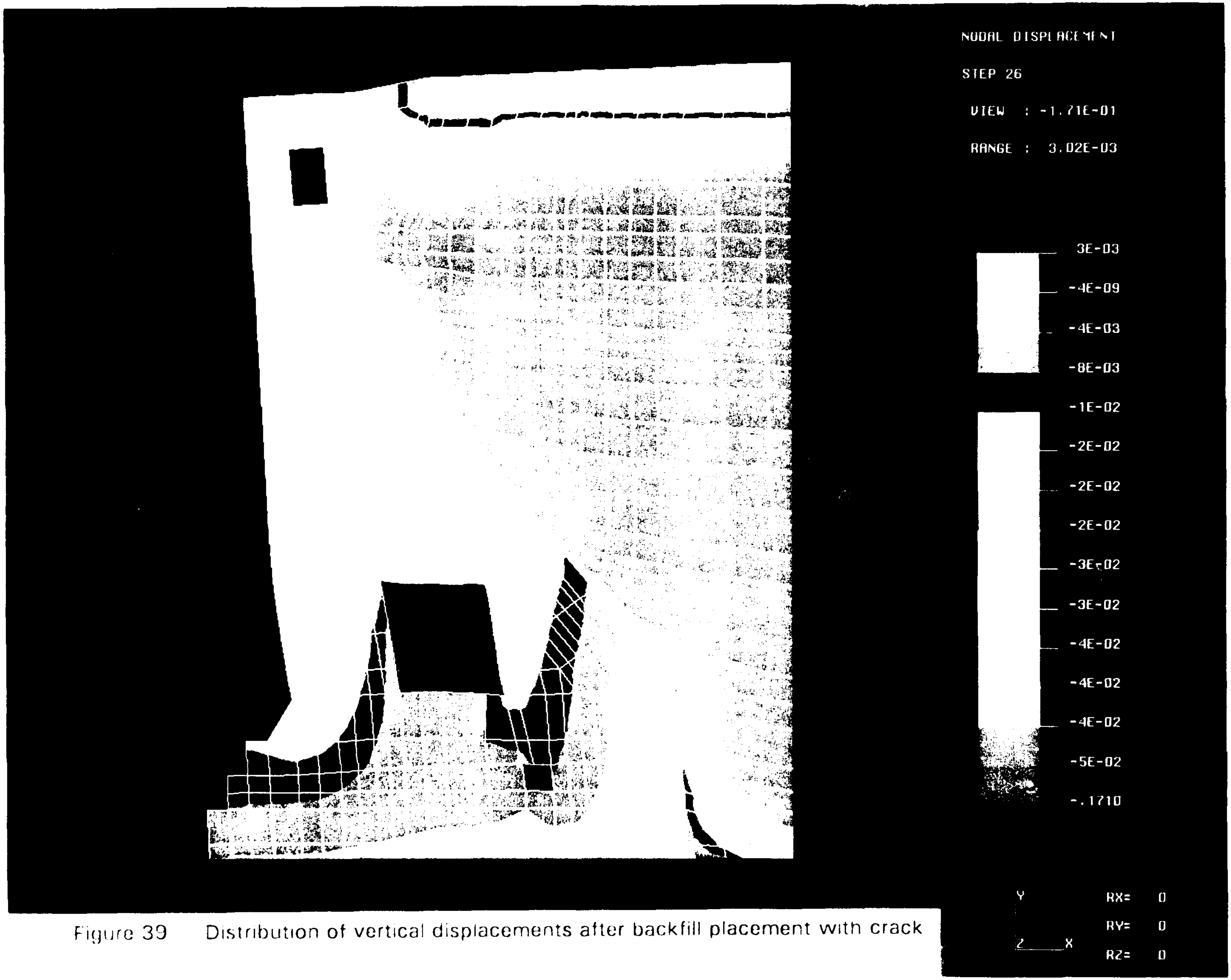


Figure 39 Distribution of vertical displacements after backfill placement with crack

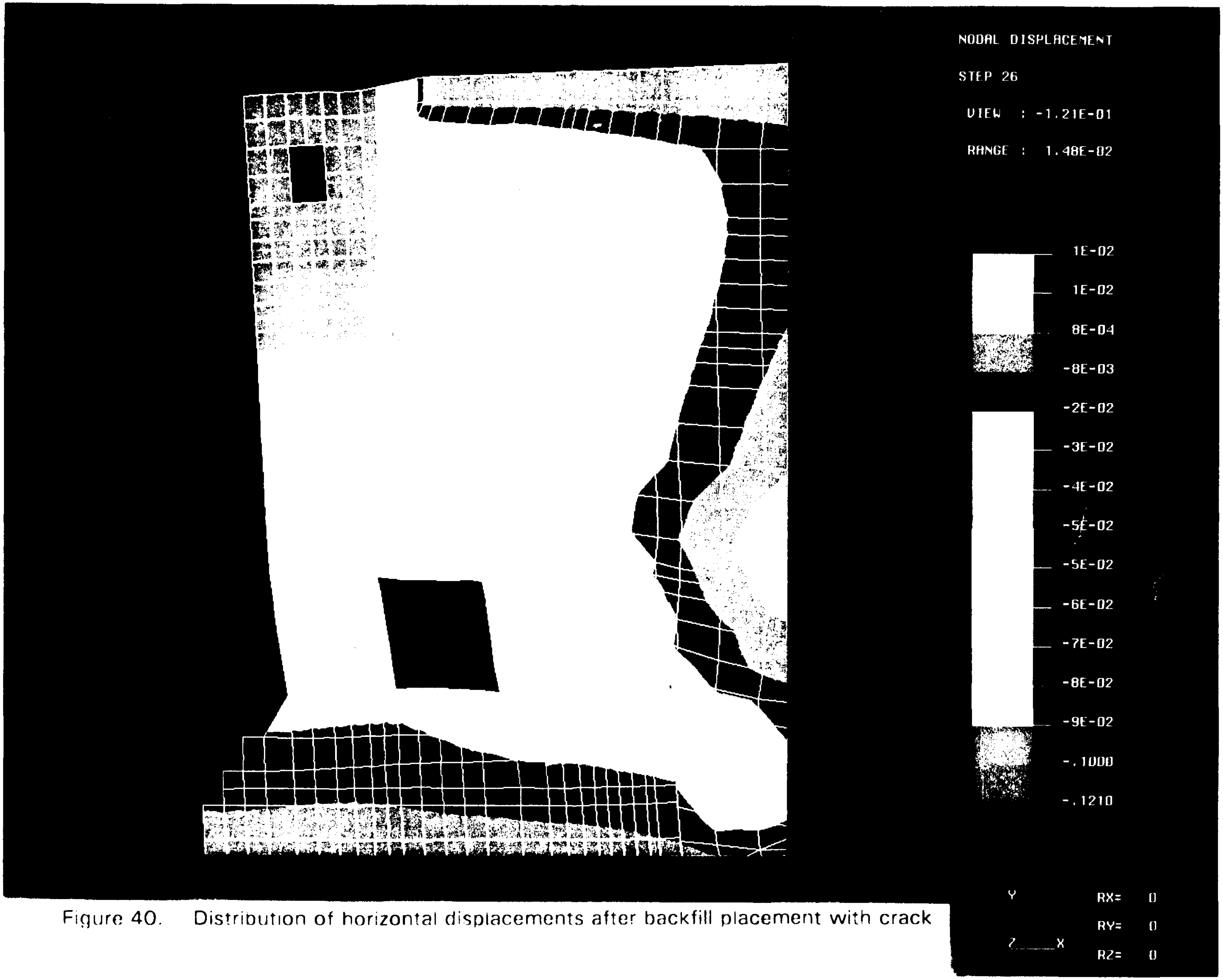


Figure 40. Distribution of horizontal displacements after backfill placement with crack

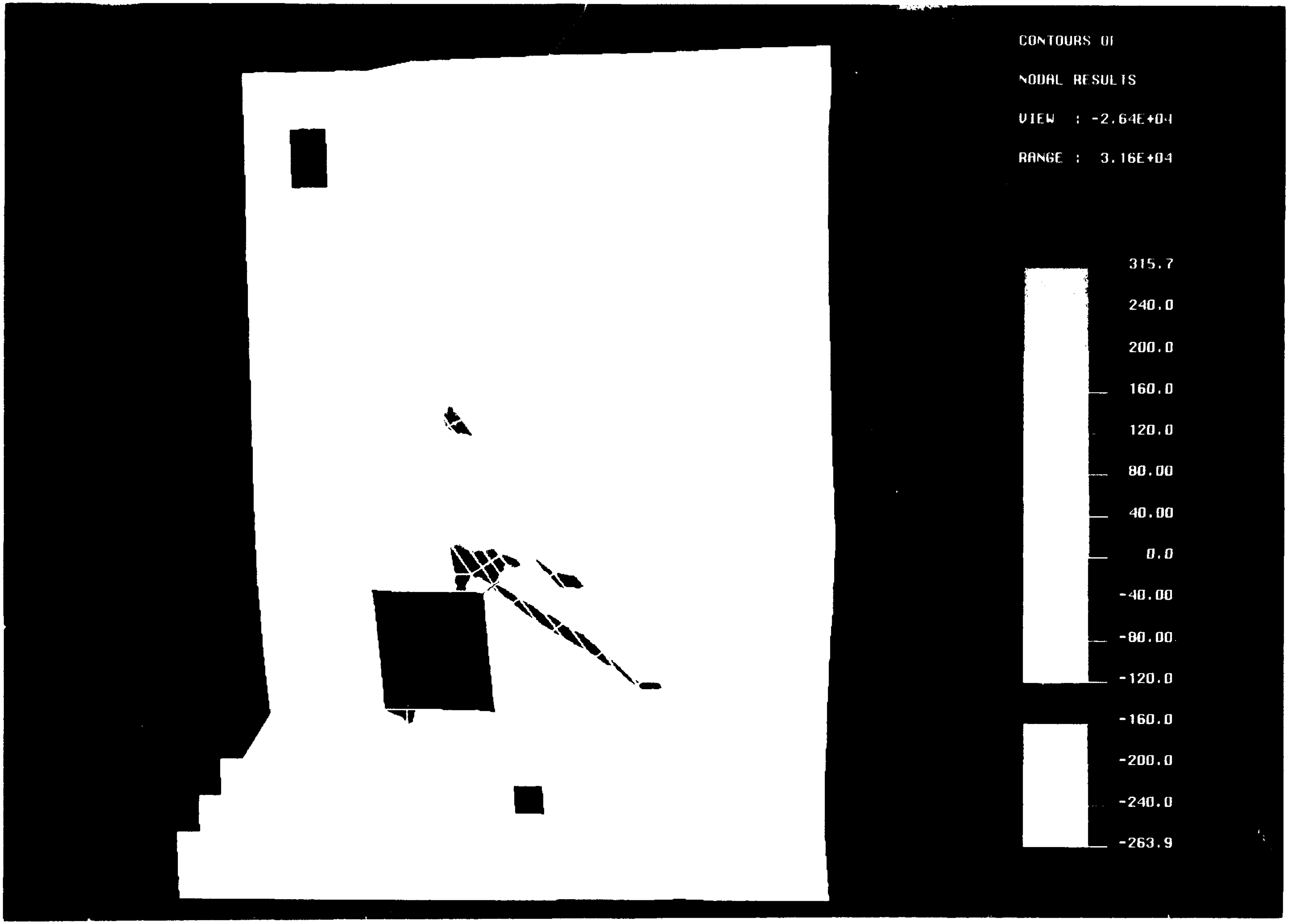


Figure 41. Distribution of minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of backfill with crack

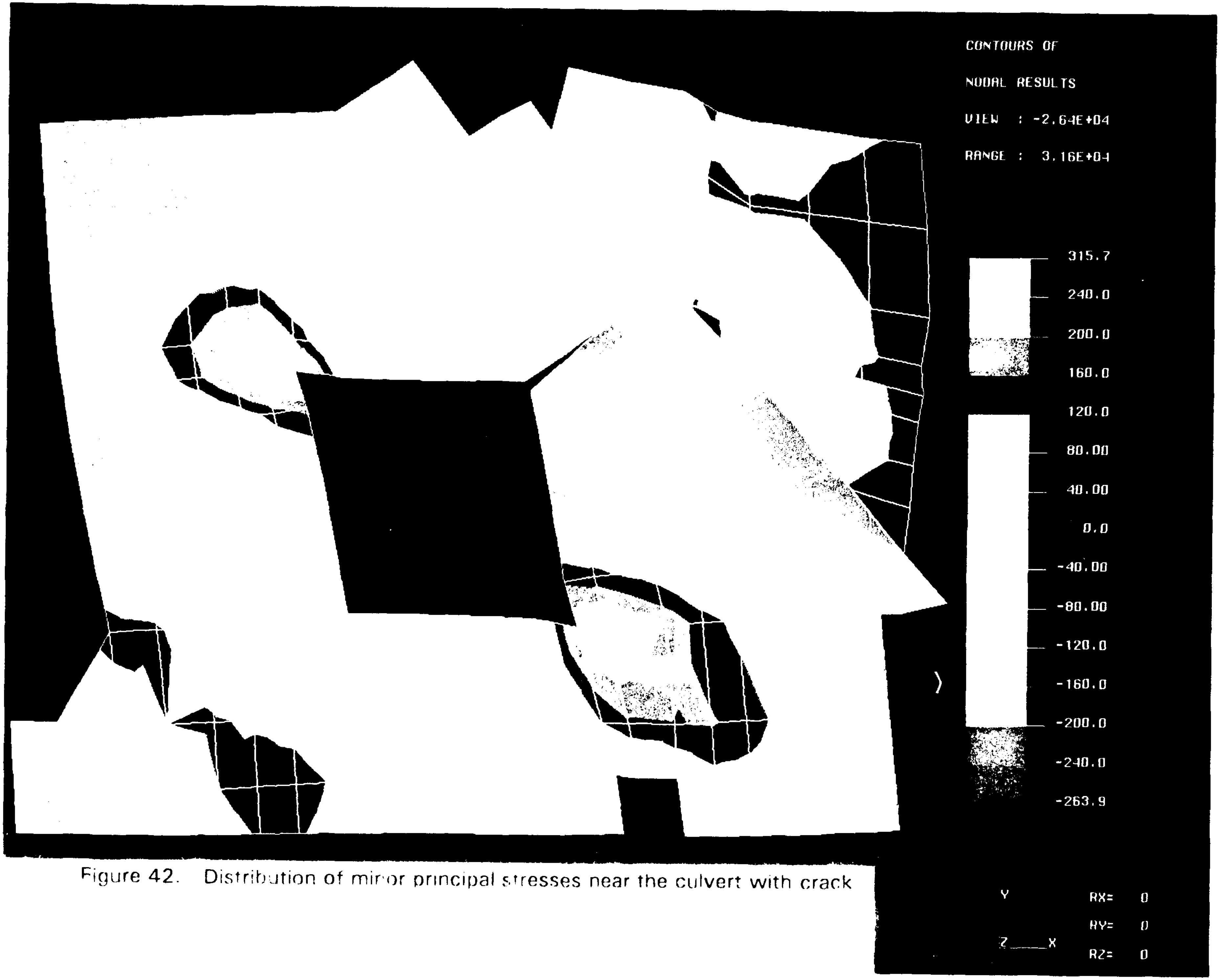


Figure 42. Distribution of minor principal stresses near the culvert with crack

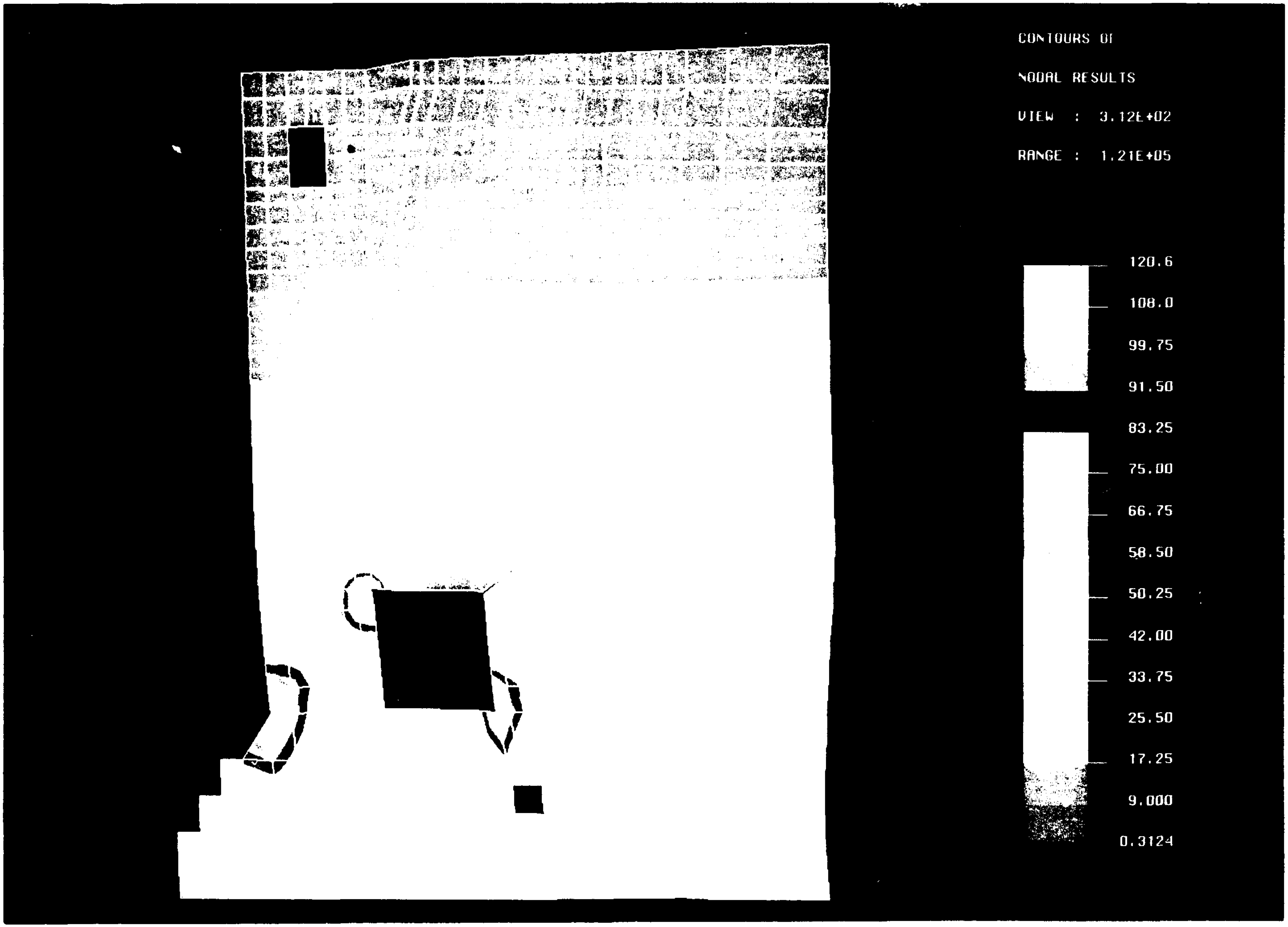


Figure 43. Distribution of major principal stresses and the deformed shape of the wall and surrounding backfill after placement of backfill with crack

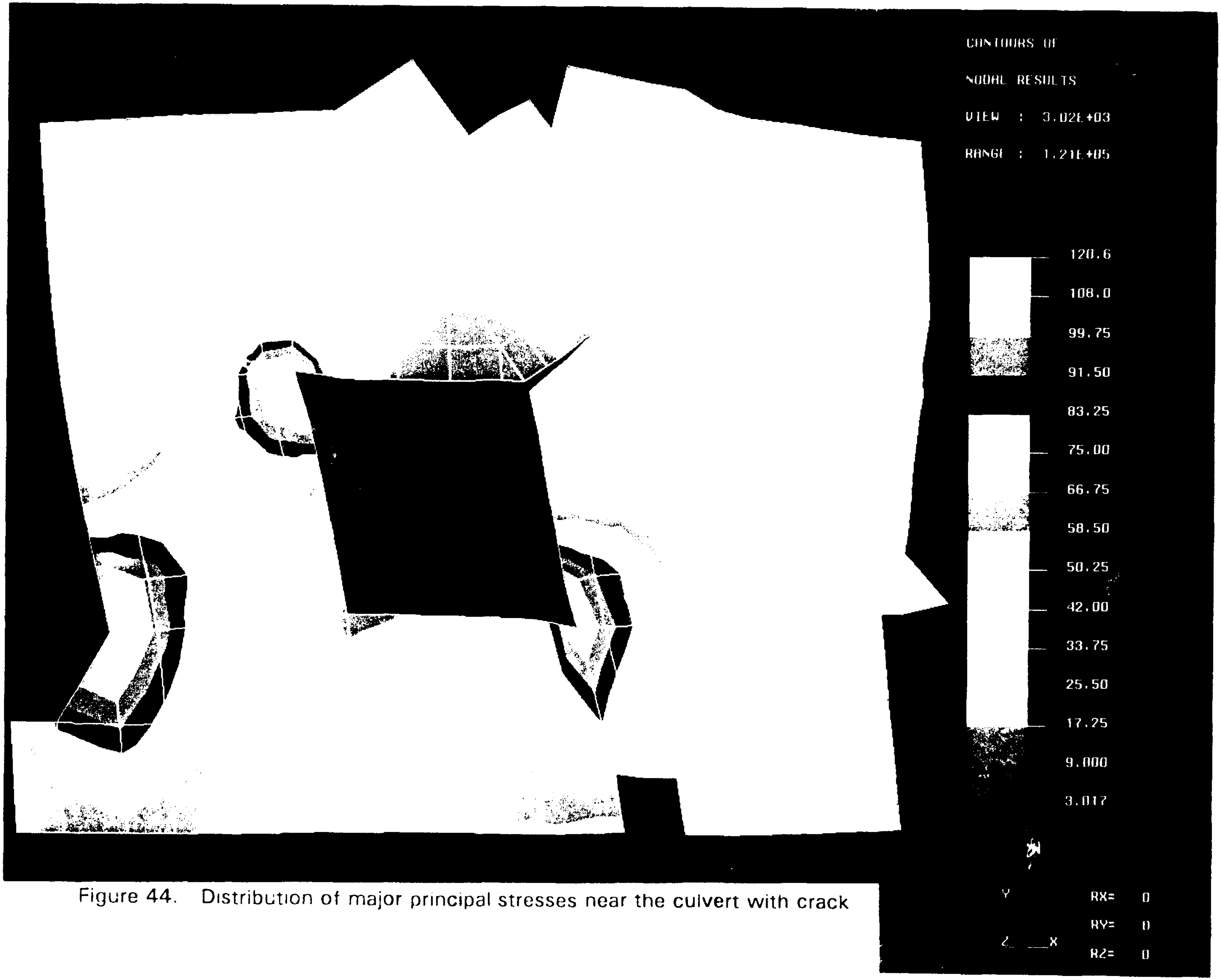


Figure 44. Distribution of major principal stresses near the culvert with crack

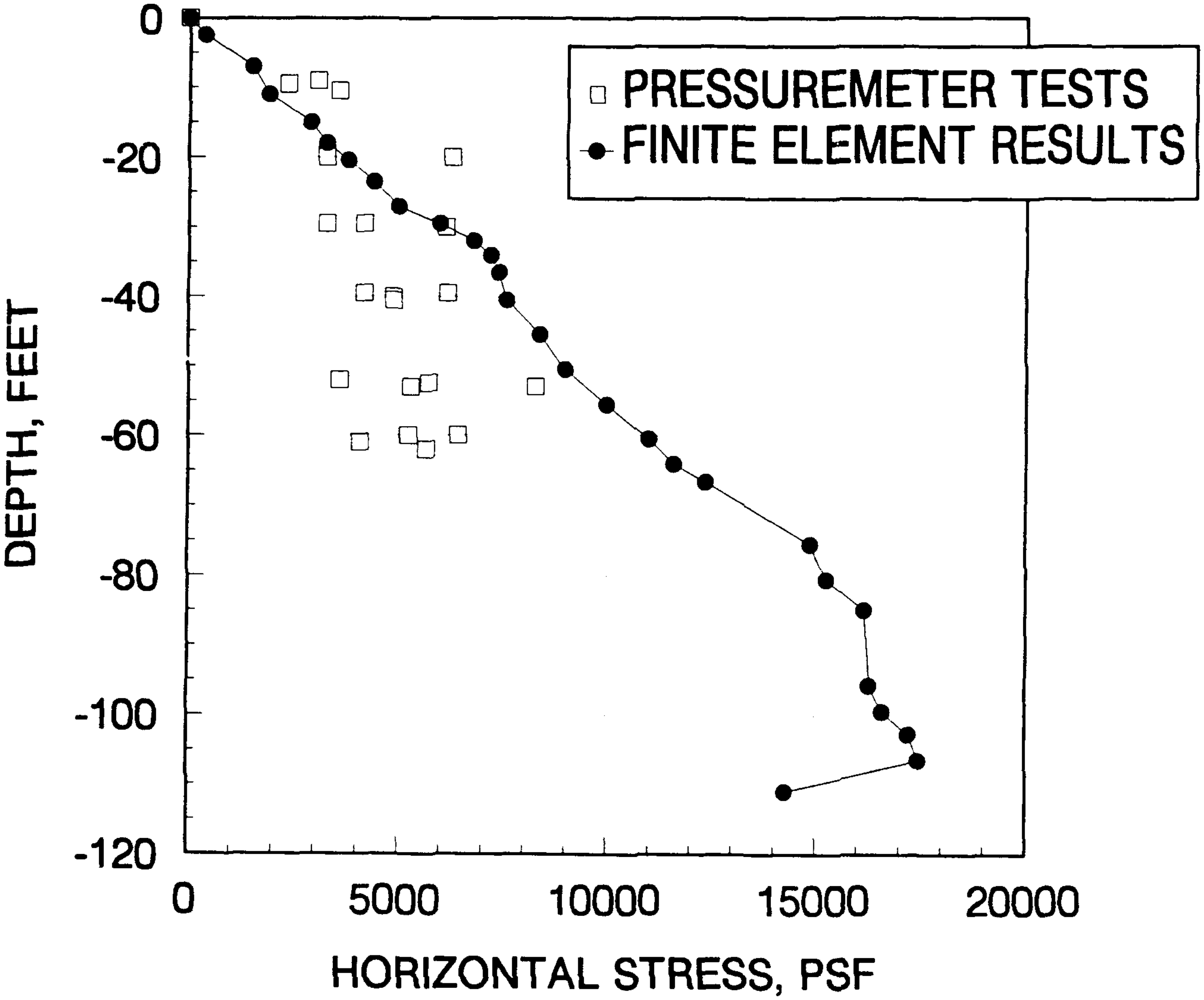


Figure 45. Lateral earth pressure on a vertical plane above the heel after backfill placement

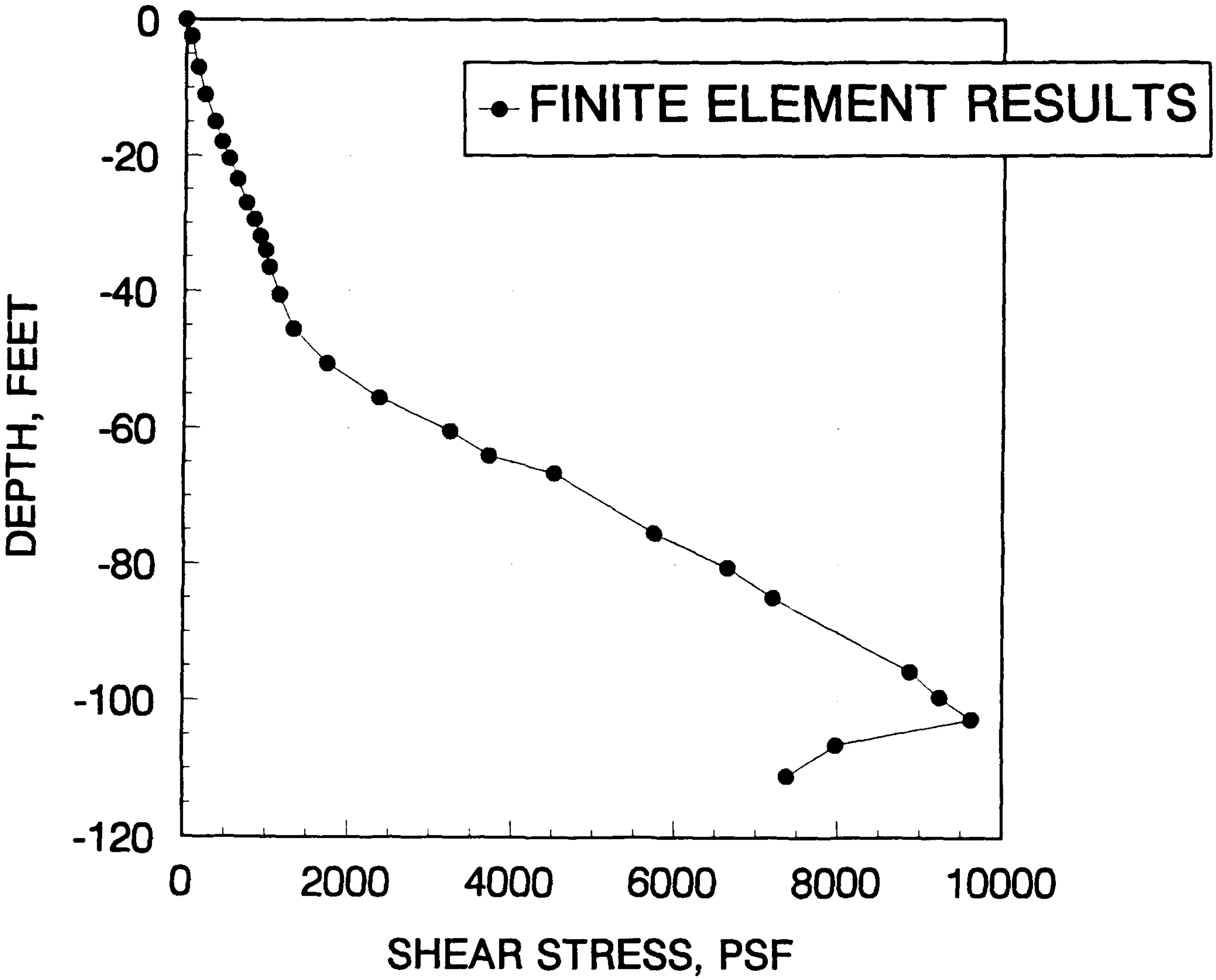


Figure 46. Shear stress on a vertical plane above the heel after backfill placement

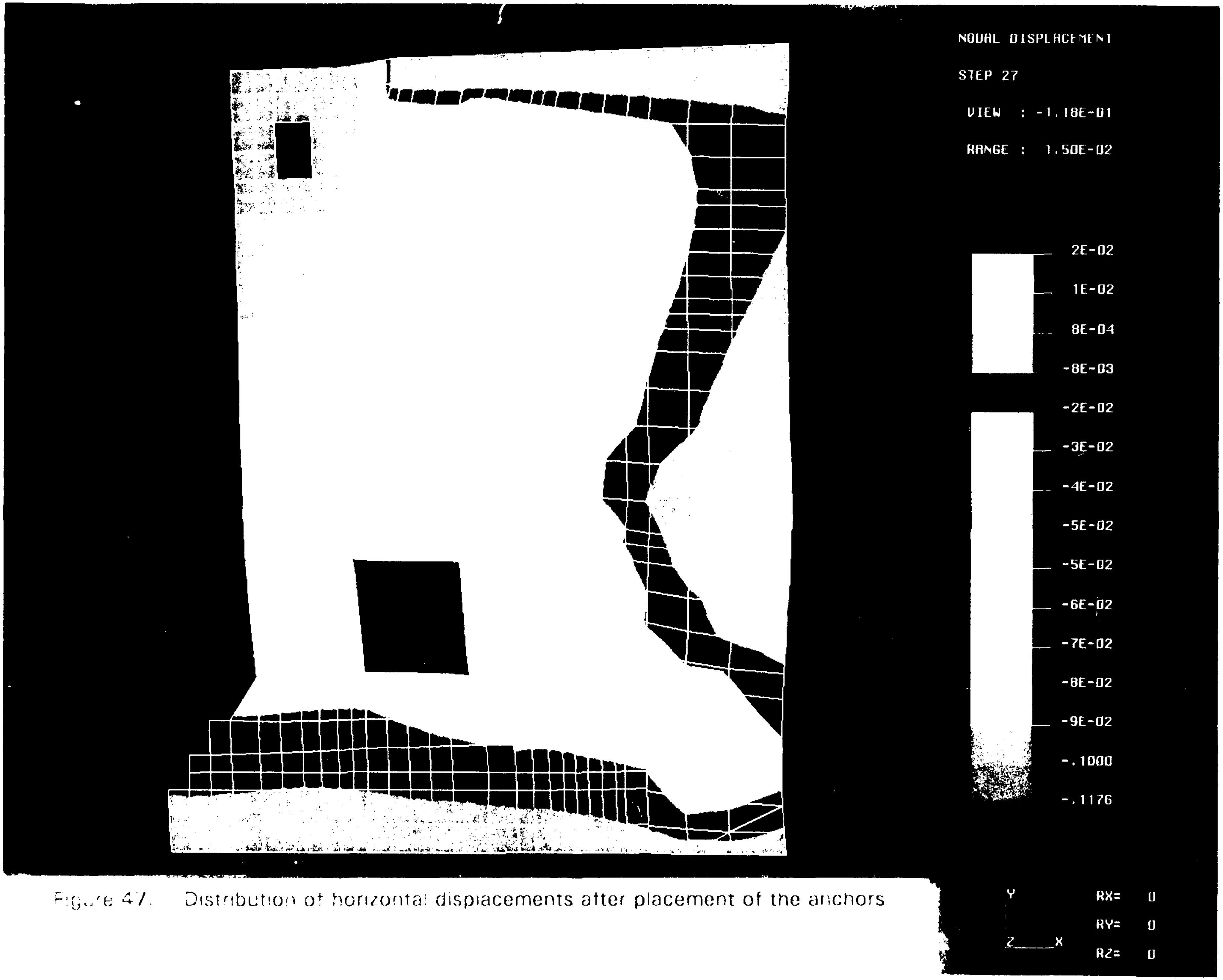
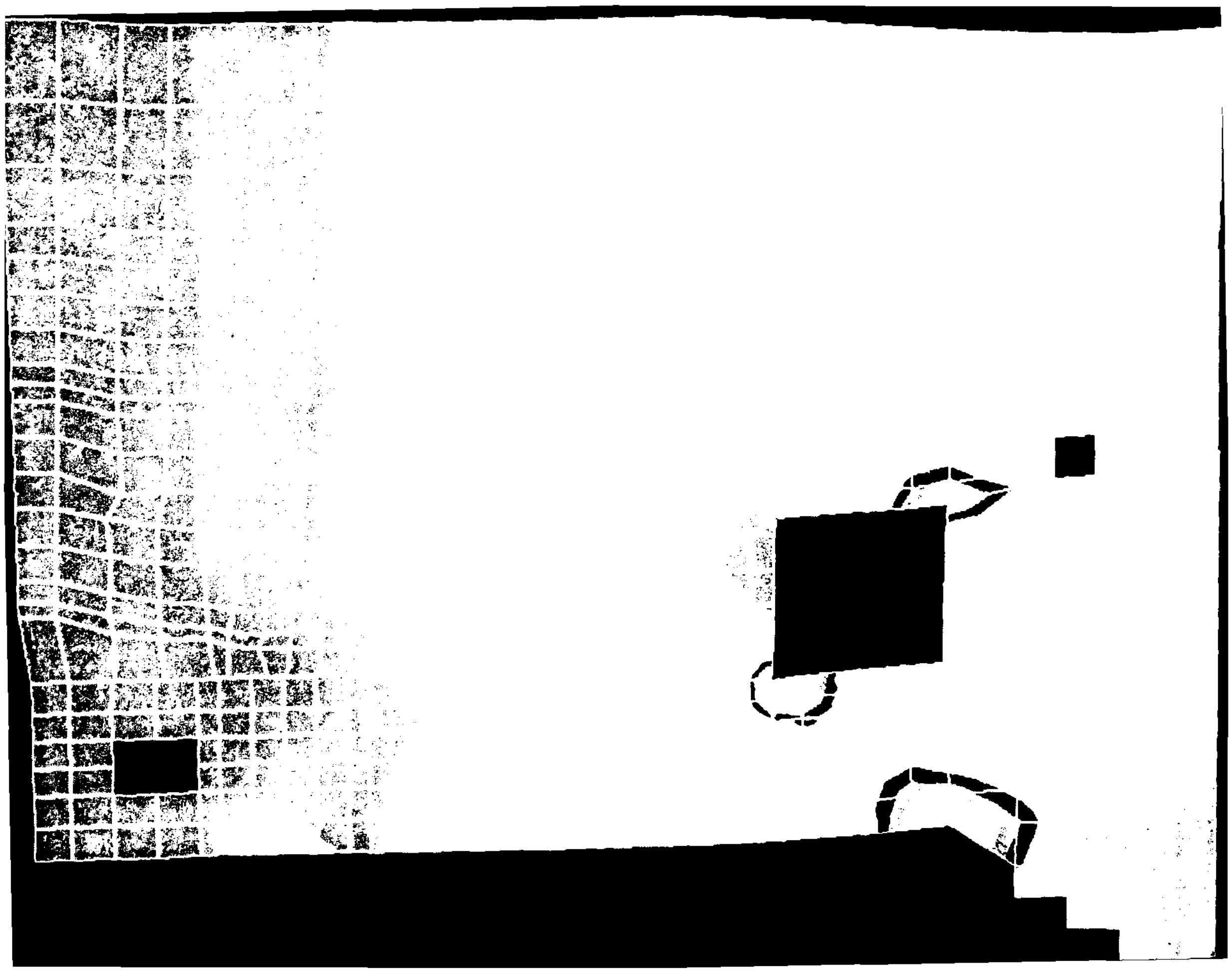
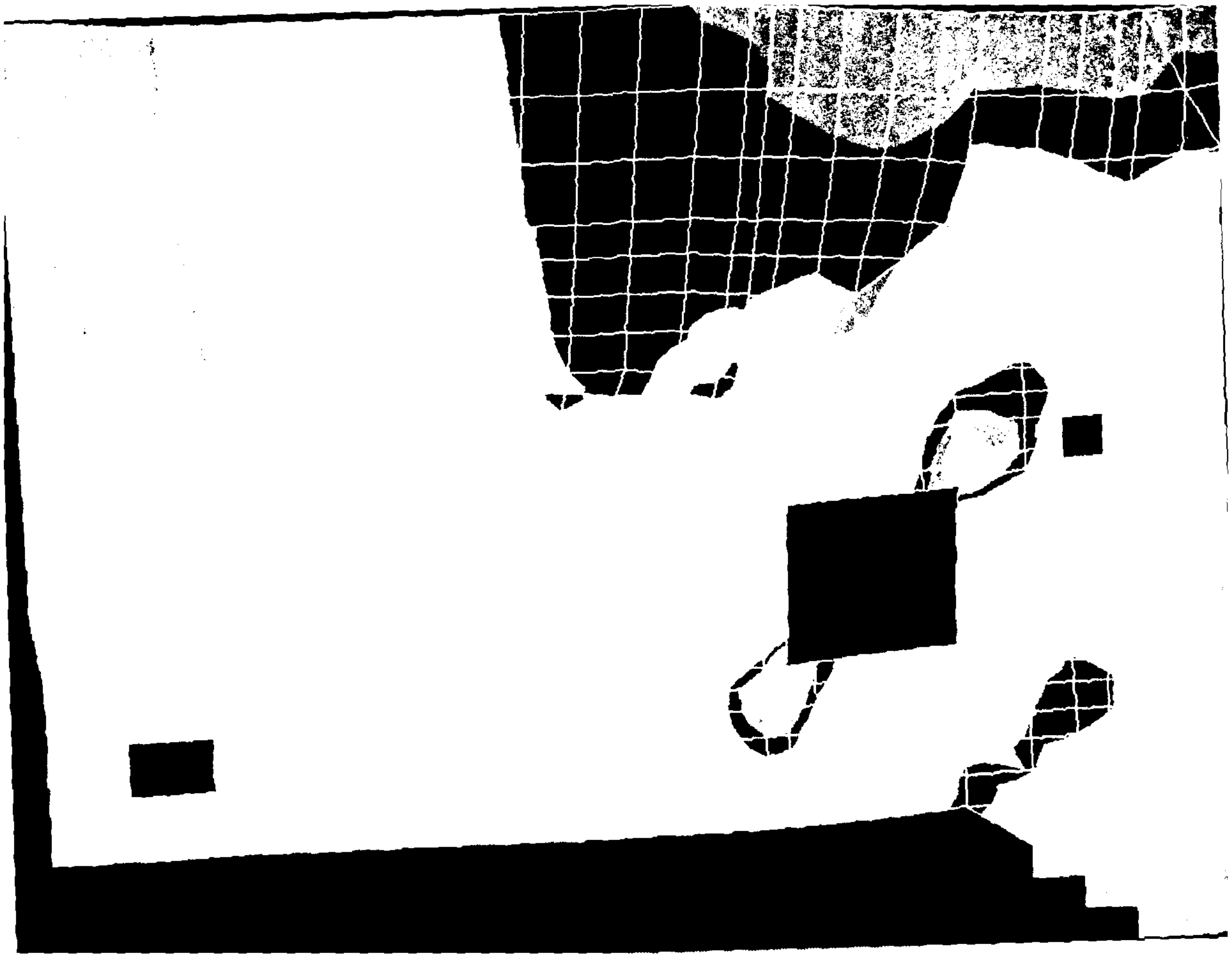


Figure 47. Distribution of horizontal displacements after placement of the anchors





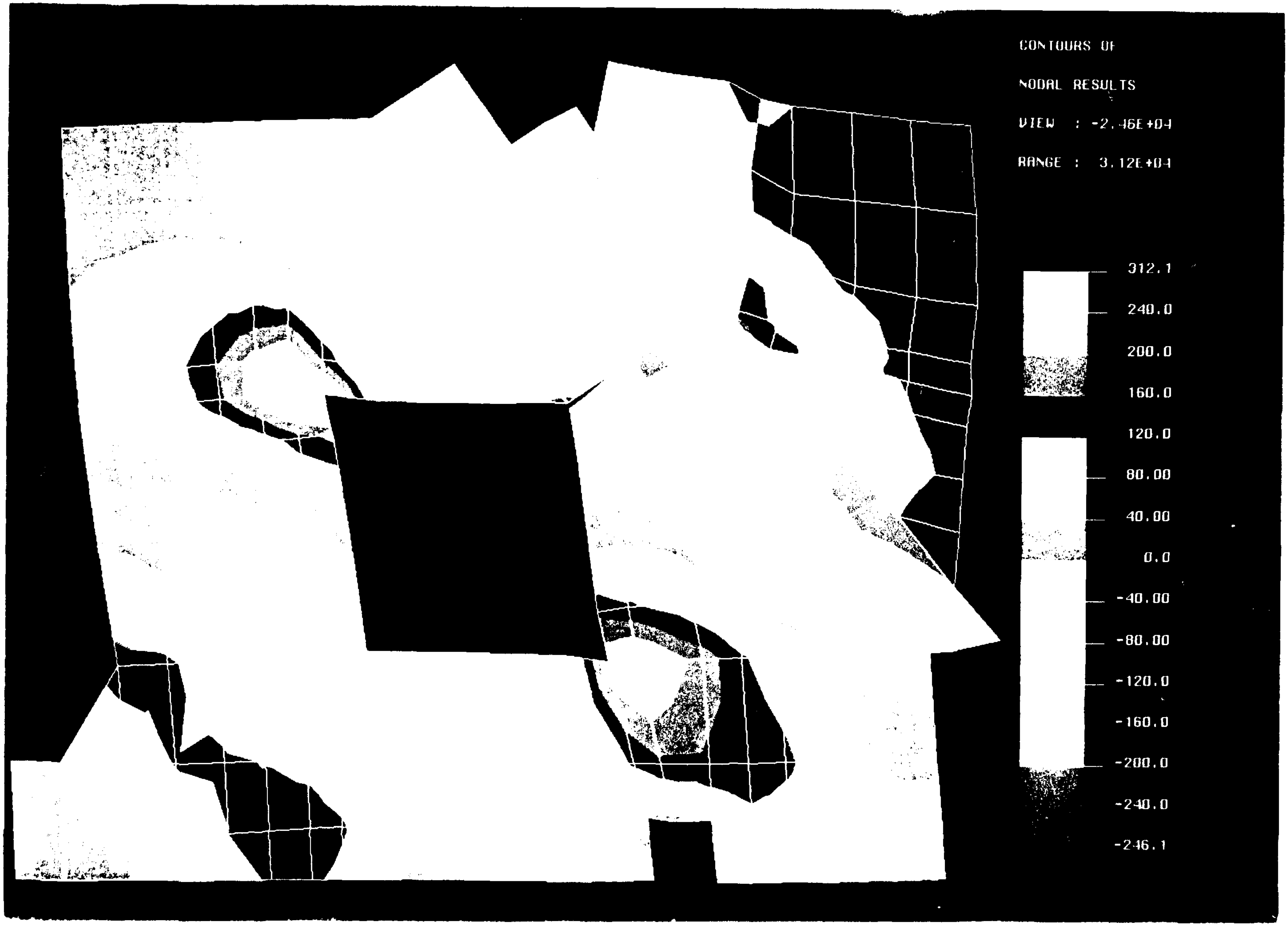


Figure 50. Distribution of minor principal stresses near the culvert after placement of the anchors

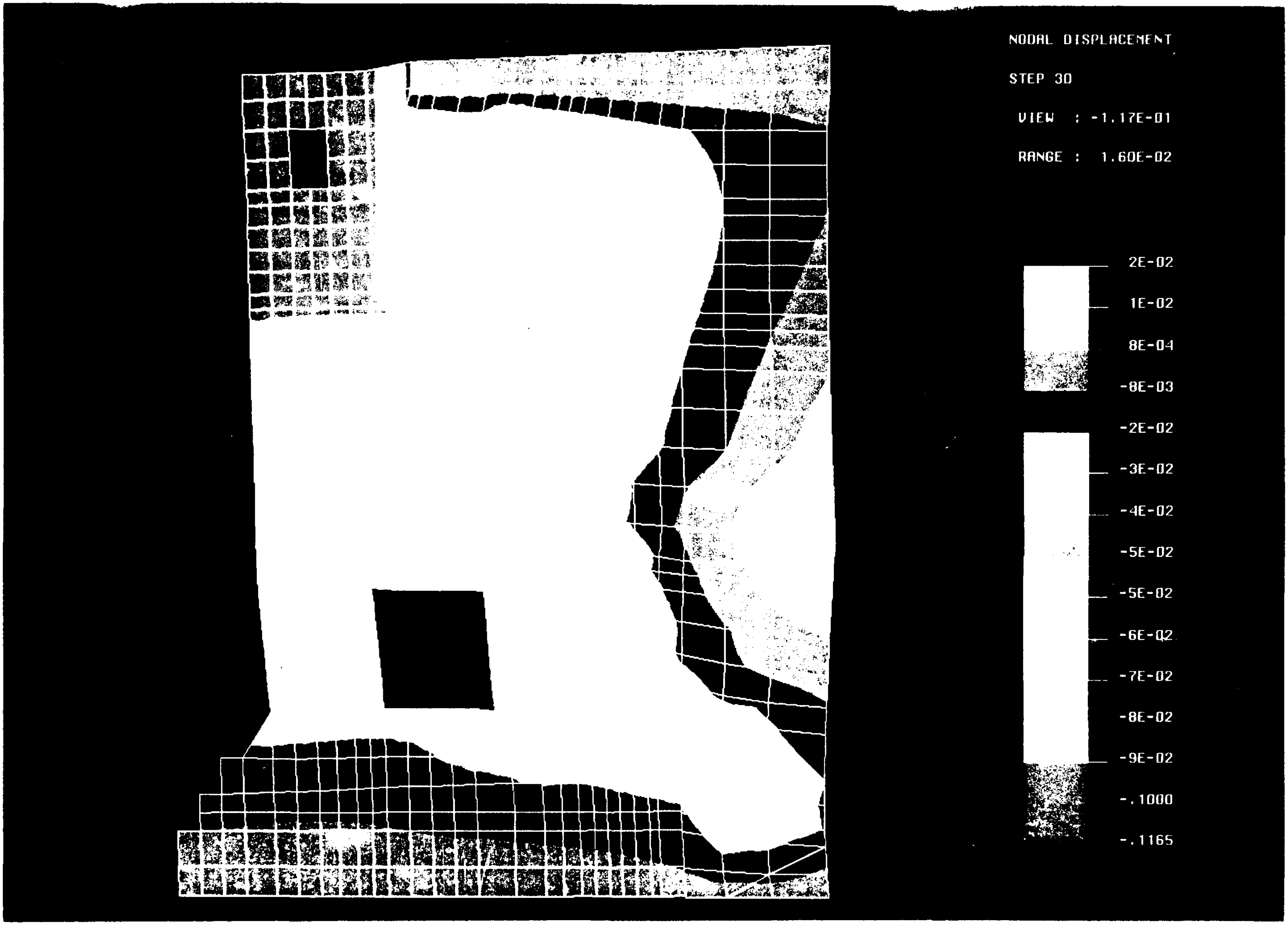


Figure 51. Distribution of horizontal displacements after raising the pool to El 200 ft

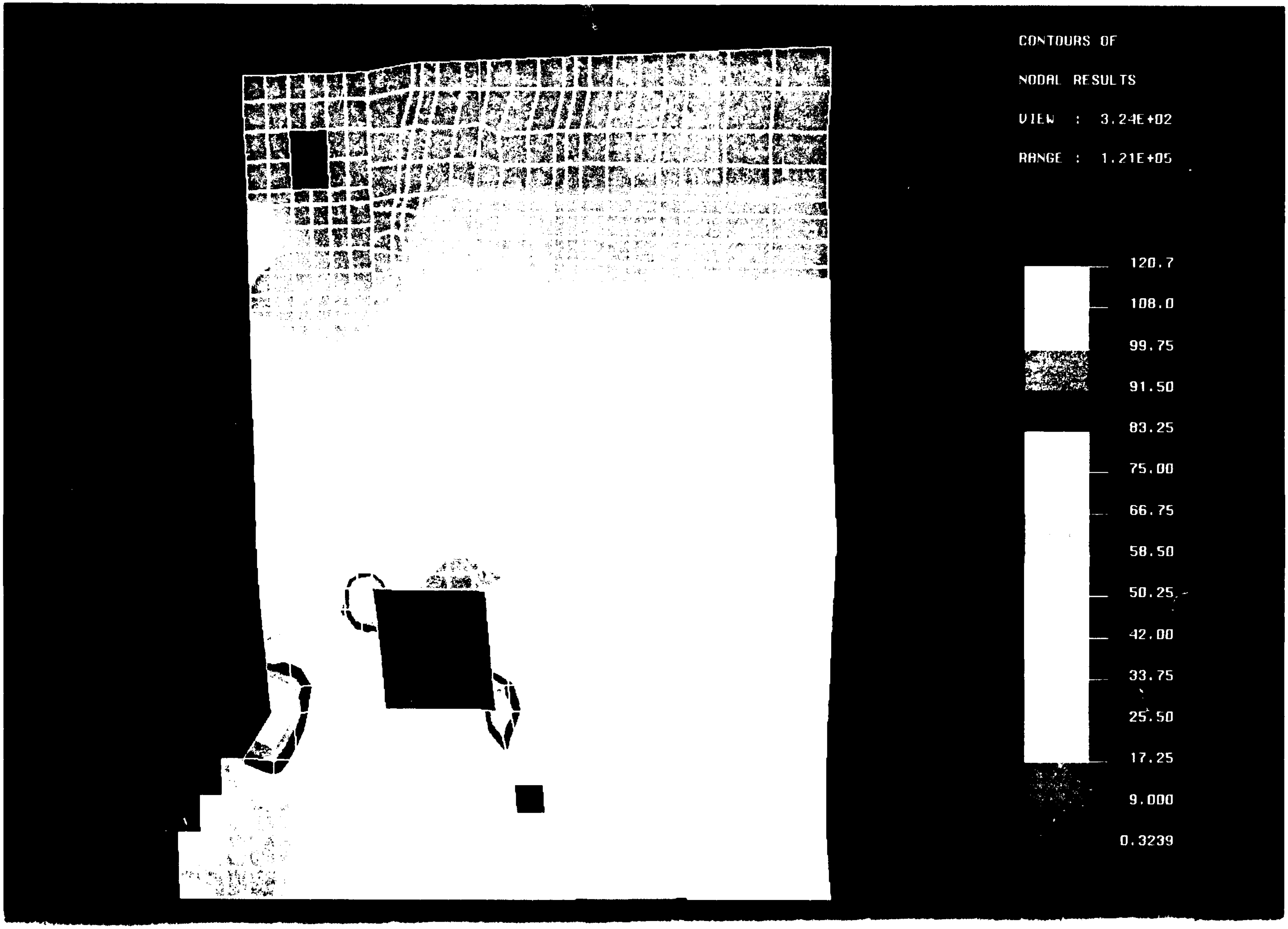


Figure 52 Distribution of major principal stresses and the deformed shape of the wall and surrounding backfill after raising the pool to El 200 ft

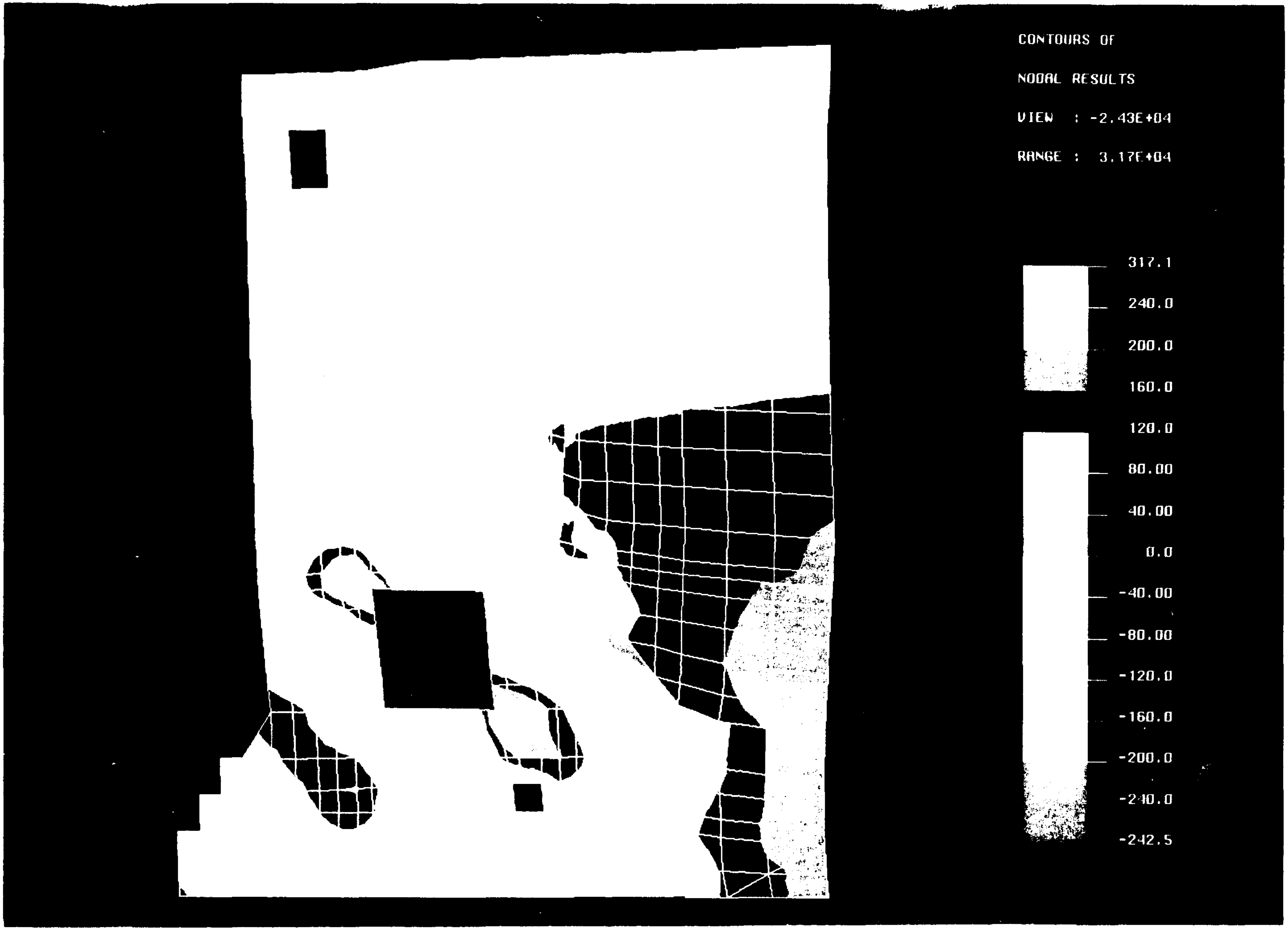


Figure 53 Distribution of minor principal stresses and the deformed shape of the wall and surrounding backfill after raising the pool to El 200 ft

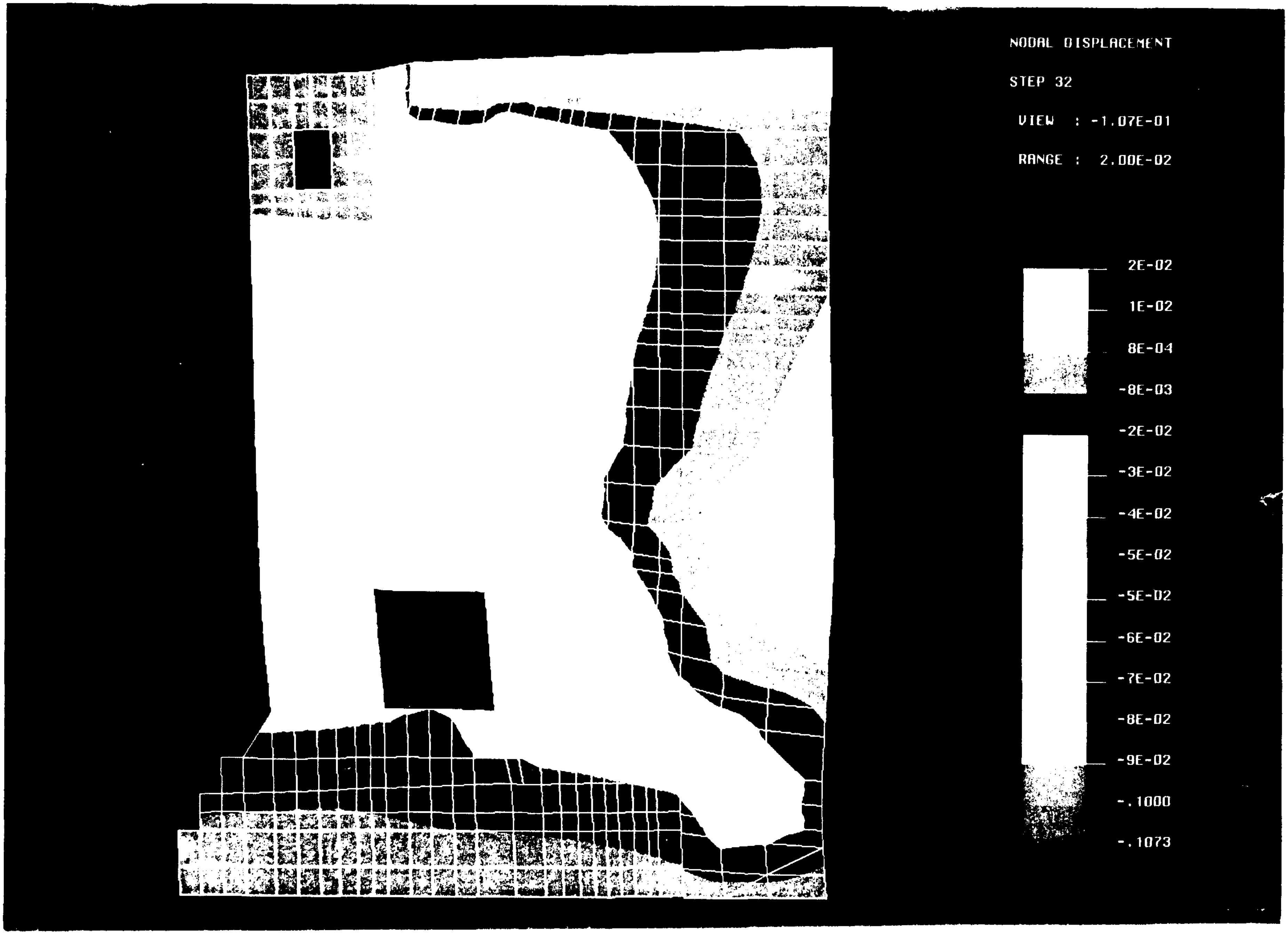
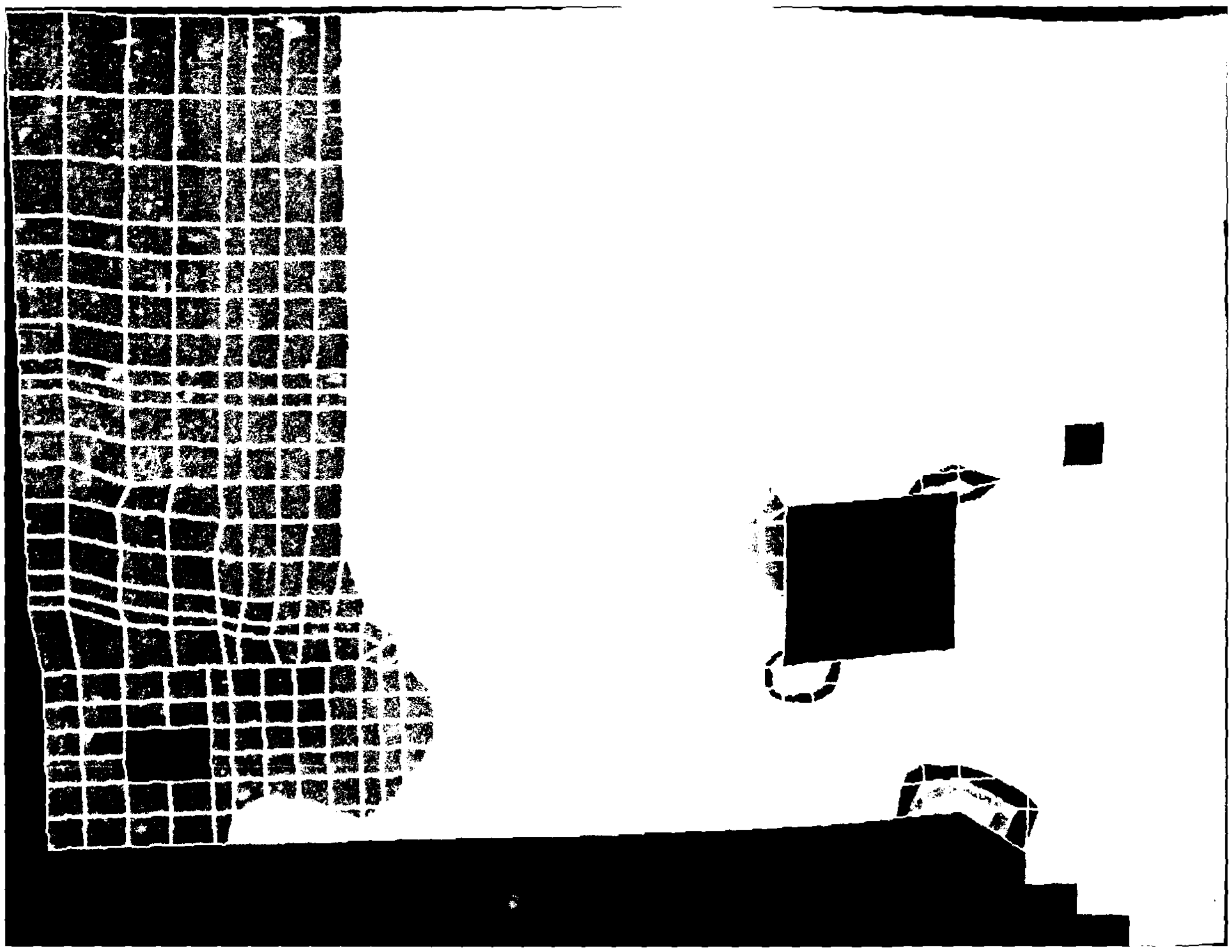
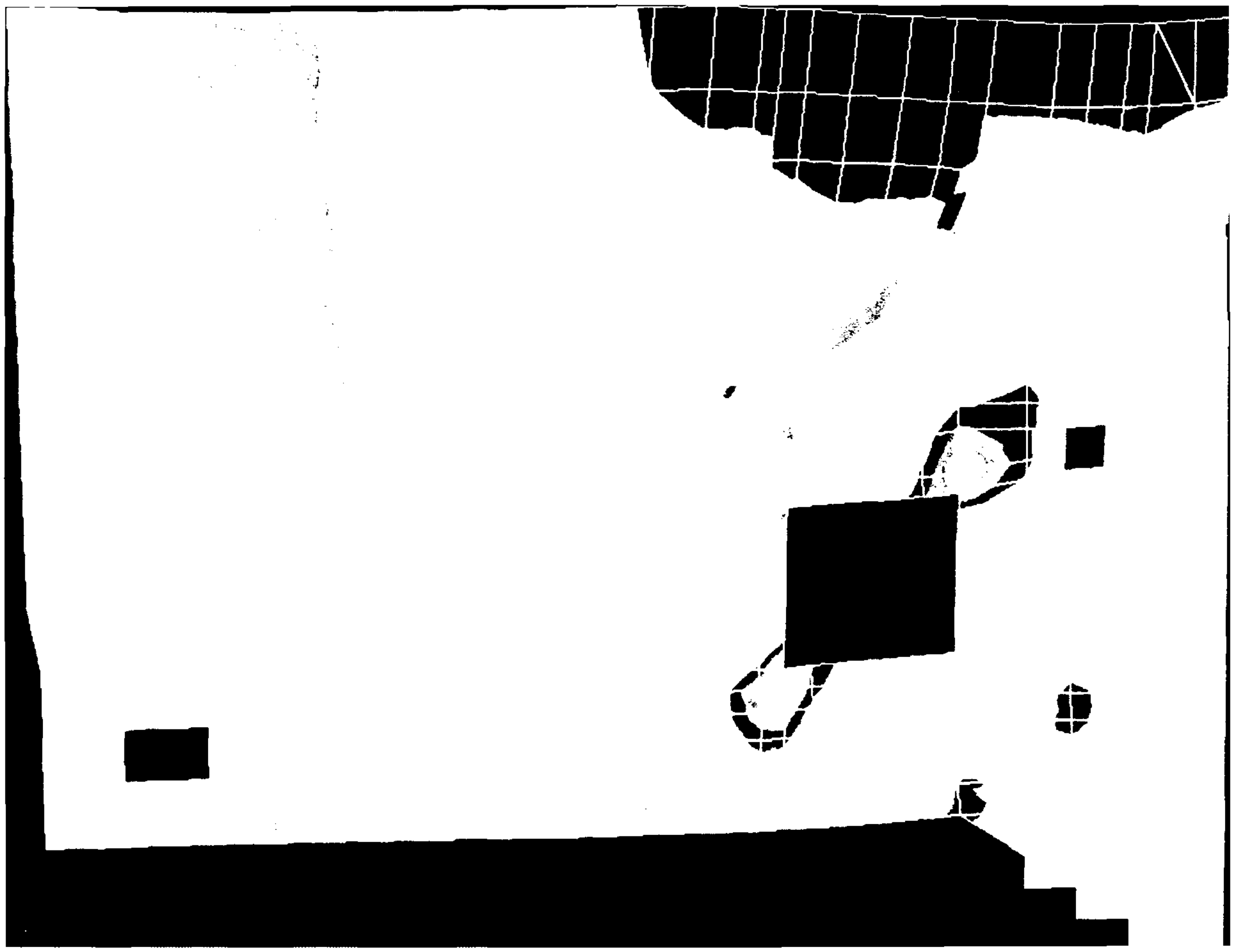


Figure 54. Distribution of horizontal displacements after raising the pool to El 240 ft





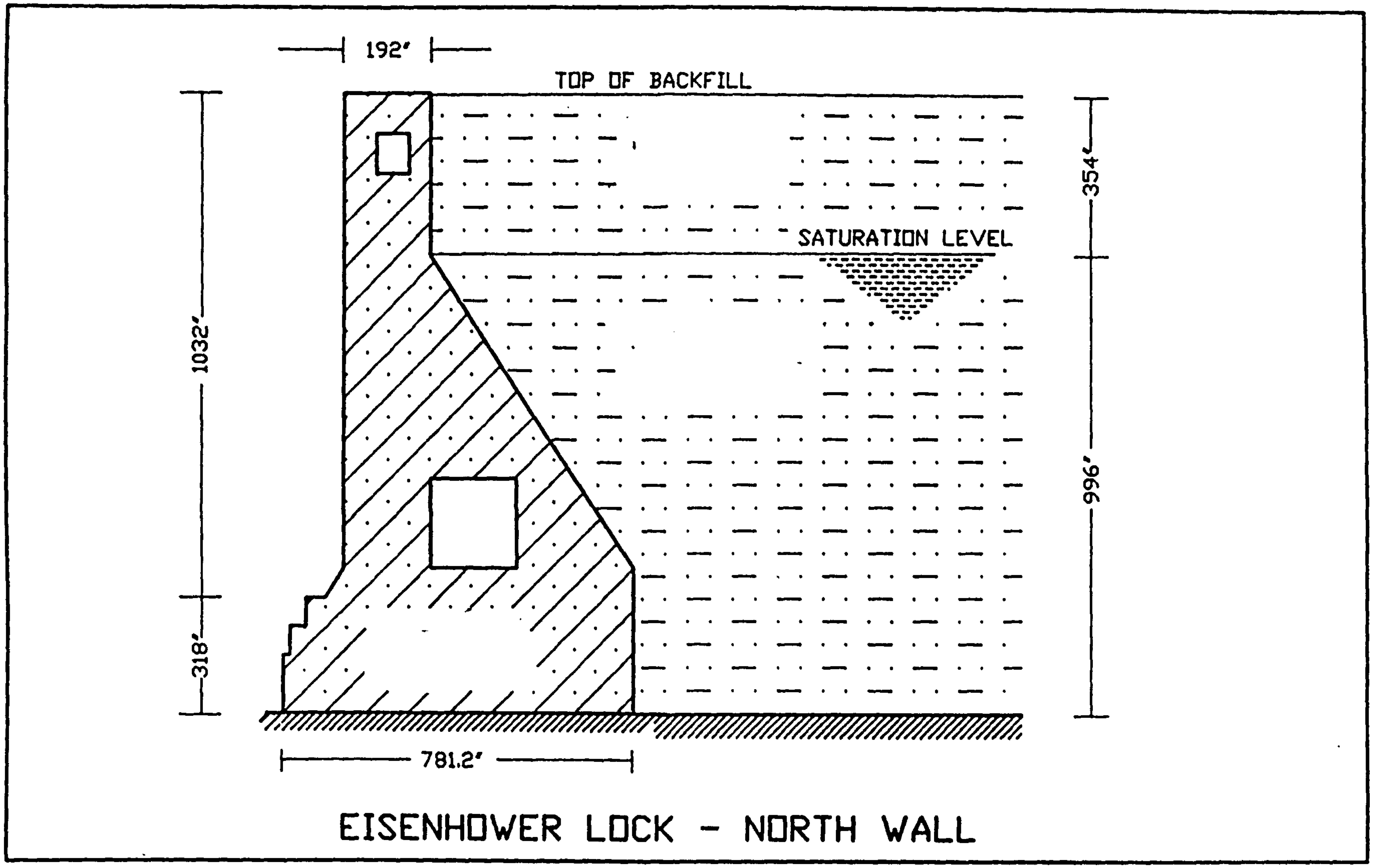


Figure 57. Typical section of the north wall of Eisenhower lock

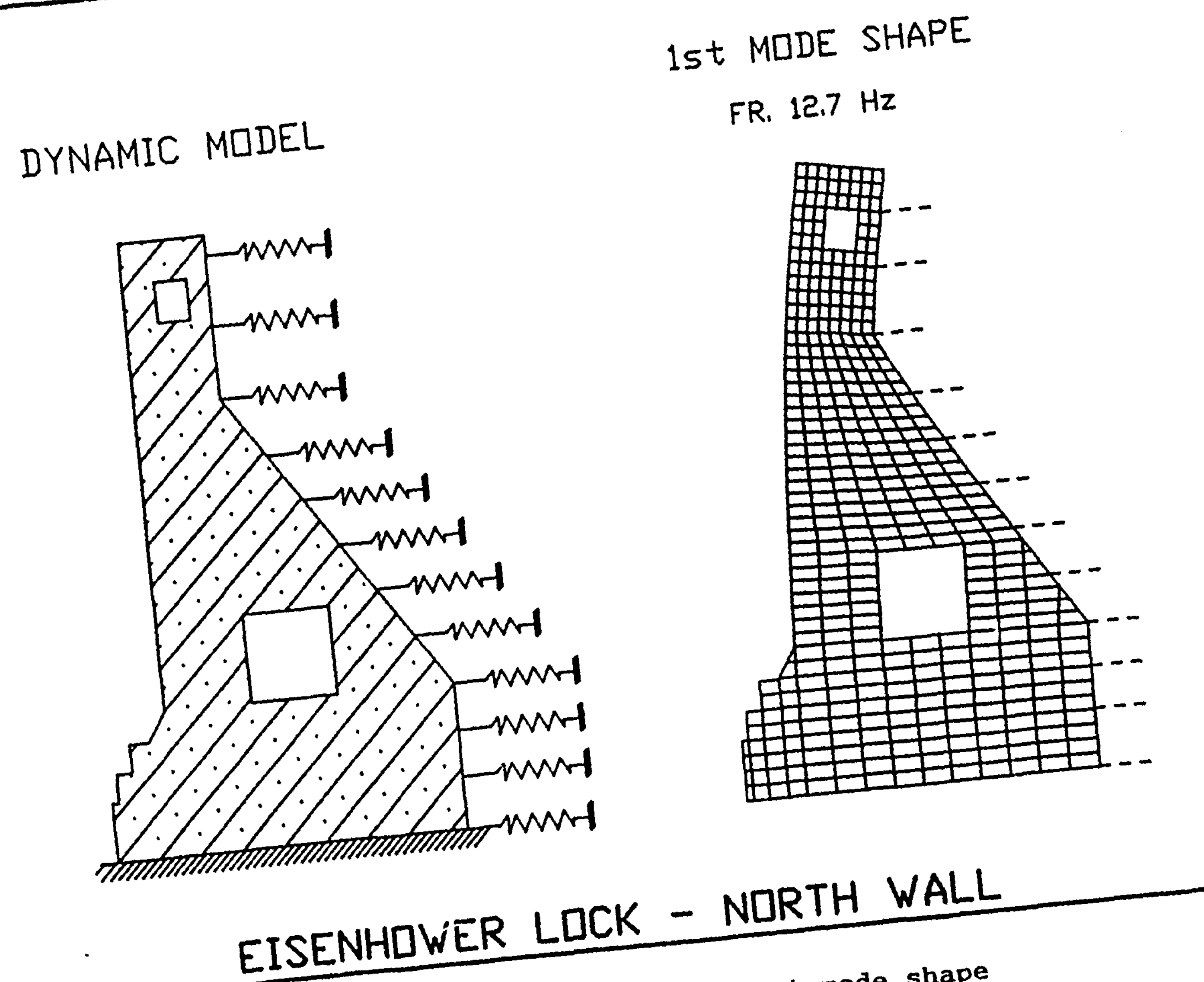


Figure 58. Finite element model and first mode shape

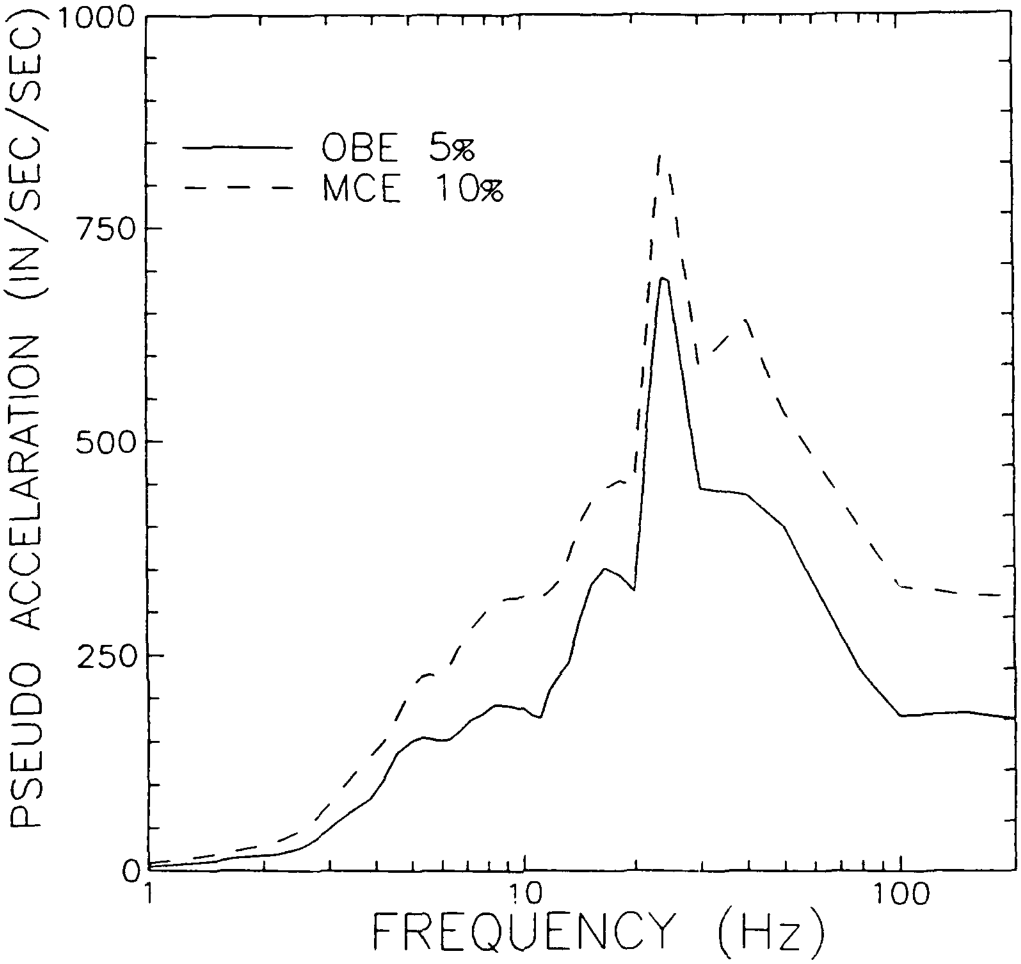


Figure 59. OBE and MCE response spectra for Eisenhower locks

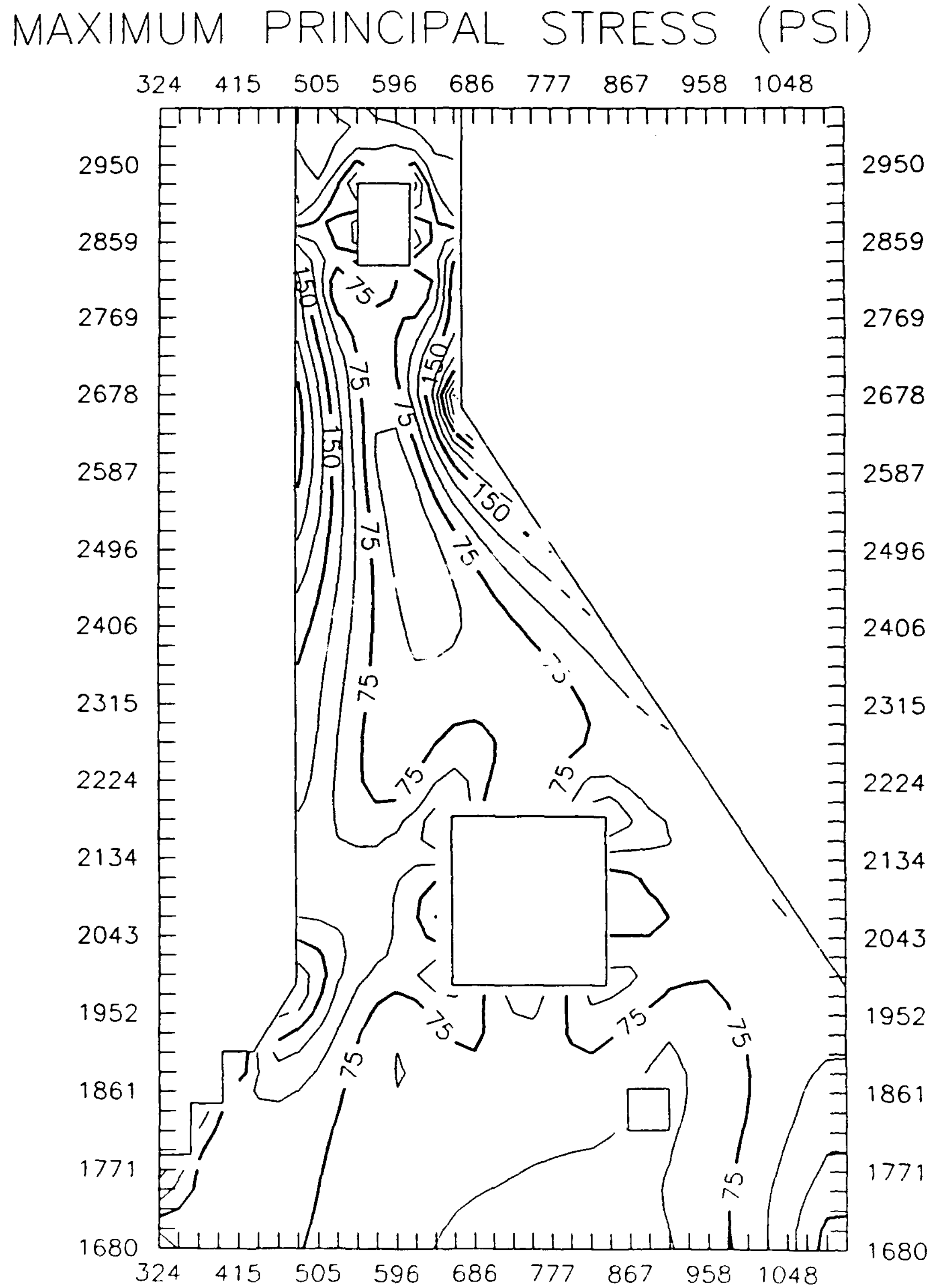


Figure 60. Principal stress contours for OBE event

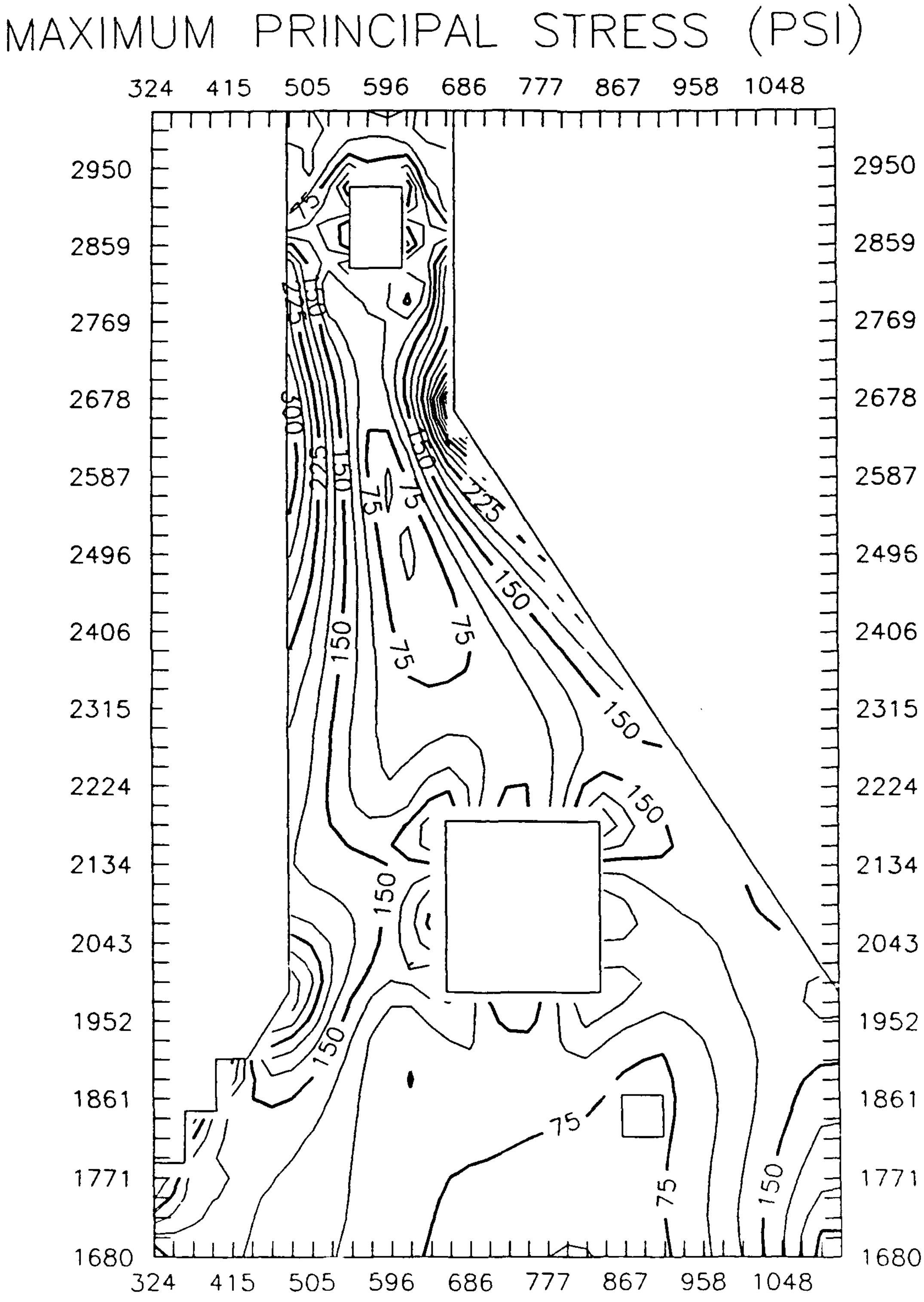


Figure 61. Principal stress contours for MCE event

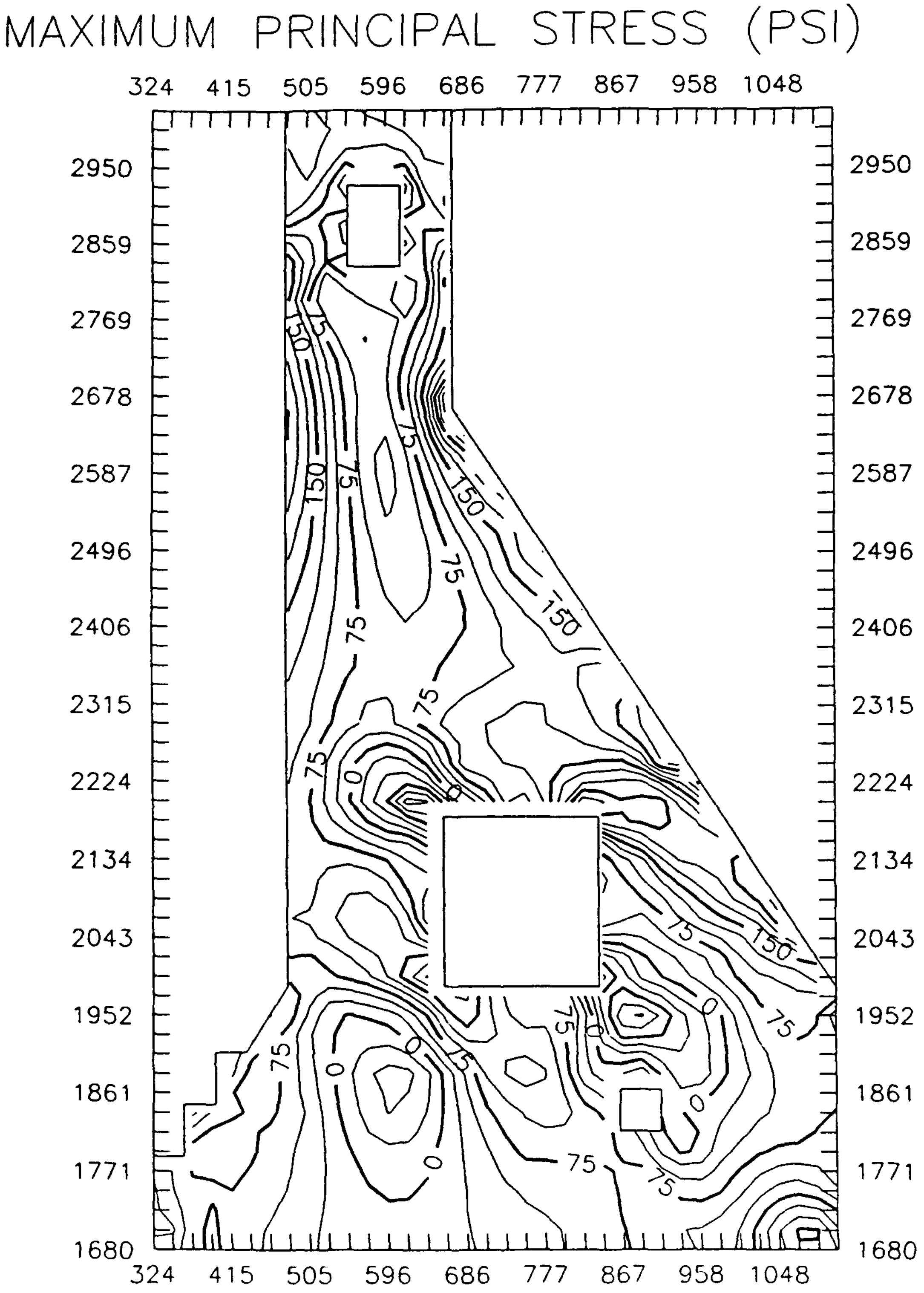


Figure 62. Static and principal stress contours for OBE event

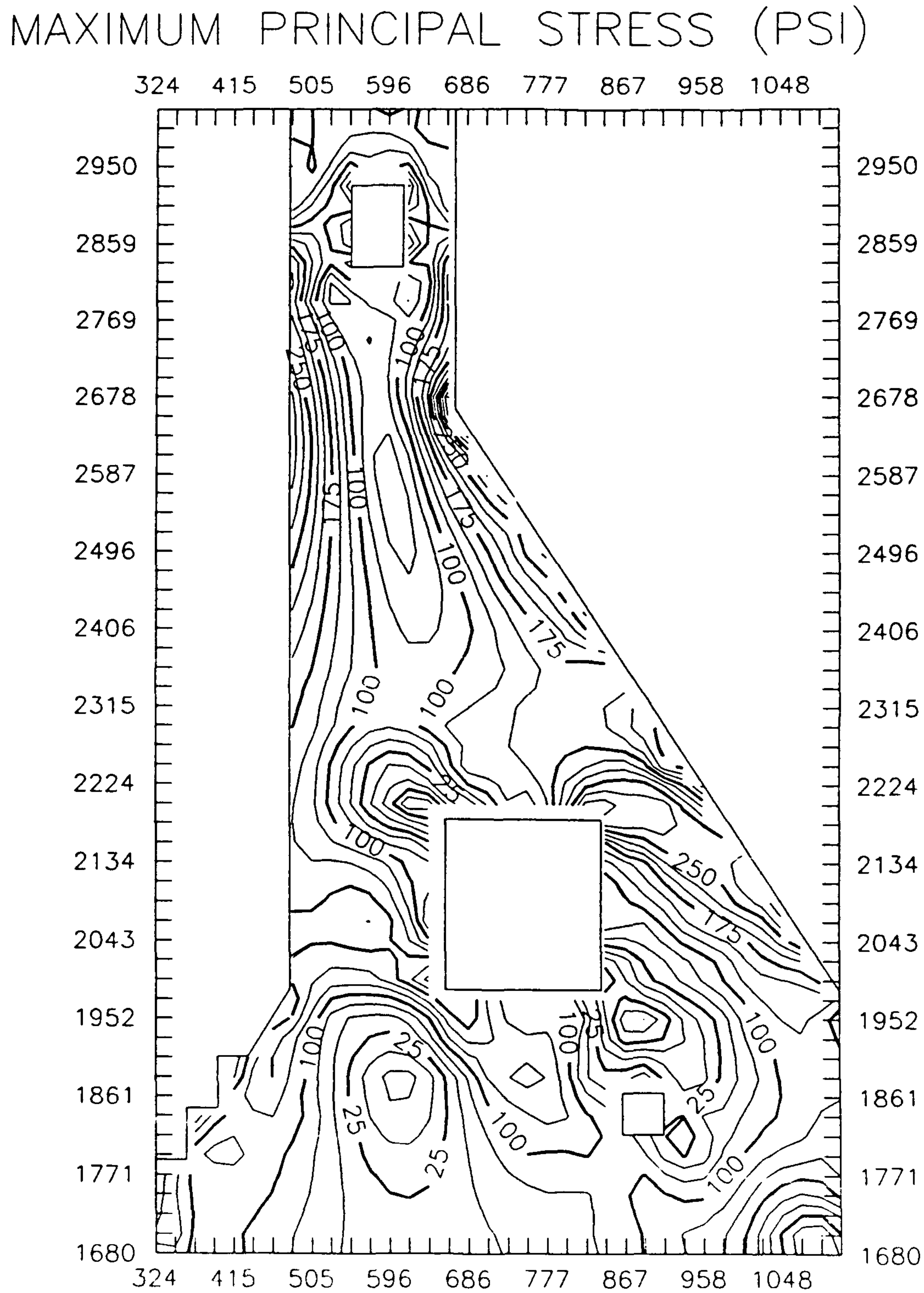


Figure 63. Static and principal stress contours for MCE event

APPENDIX A: INTERGOVERNMENTAL AGENCY AGREEMENT

CEWES IM-DI

APR 12 1990 (Reed Mosher)

Study Management/Project Engineering Branch

SUBJECT: Structural Evaluation, Eisenhower/Snell Locks

Mr. Steven Hung
Contracting Officer's Representative
Saint Lawrence Seaway Development Corporation
Office of Engineering and Planning
Post Office Box 520
Massena, New York 13662-0520

Dear Mr. Hung:

Enclosed is the final Scope of Work for the subject study. The Waterways Experiment Station (WES) will be performing the bulk of the work for this study. Their Detailed Scope of Work is incorporated into the final Scope of Work. The Mobile District of the Corps of Engineers will be doing the core drilling and sampling under the direction of WES. The Buffalo District will be preparing the final report and is also responsible for study coordination and overall project management. The North Central Division (NCD) of the Corps in Chicago and the Office of the Chief of Engineers (OCE) in Washington are providing technical assistance.

The study cost is estimated at $570,000. A detailed cost estimate and study schedule are contained in the enclosed scope.

The study is proceeding in accordance with the schedule contained in the interagency agreement between your agency and the Corps of Engineers. The agreement was signed on February 2, 1990 by General Patin. I anticipate on-site work will commence on May 15, 1990. My project manager will provide you a monthly report on the progress of the study.

Study Management/Project Engineering Branch
SUBJECT: Structural Evaluation, Eisenhower/Snell Locks

My point of contact pertaining to this matter is Mr. Michael Barton, P.E., of the Study Management/Project Engineering Branch who may be contacted at 716-879-4231 or by writing to him at the above address.

Sincerely,
BRUCE W. HAIGH
LTC. U.S. ARMY
COMMANDER
Bruce W. Haigh
Lieutenant Colonel, U.S. Army
Acting District Commander

Enclosure

CF:
CECW-EG (Tony Liu)
CENCD-ED-TT (Joe Schmidt)
CEWES-IM-DI (Reed Mosher)

Barton/mb__________
Gorecki__________
Nicaise__________
Gilbert__________
Brooks__________
Haigh__________

STRUCTURAL EVALUATION OF EISENHOWER AND SNELL LOCKS

SAINT LAWRENCE SEAWAY

MASSENA, NEW YORK

FINAL SCOPE OF WORK

PREPARED FOR

SAINT LAWRENCE SEAWAY DEVELOPMENT CORPORATION

MASSENA, NEW YORK

PREPARED BY

U.S. ARMY CORPS OF ENGINEERS
BUFFALO DISTRICT

APRIL 1990

STRUCTURAL EVALUATION OF EISENHOWER AND SNELL LOCKS

SAINT LAWRENCE SEAWAY

MASSENA, NEW YORK

TABLE OF CONTENTS

STRUCTURAL EVALUATION OF EISENHOWER AND SNELL LOCKS

SAINT LAWRENCE SEAWAY

MASSENA, NEW YORK

1. PURPOSE AND SCOPE

The purpose of the structural evaluation of Eisenhower and Snell Locks is to determine the adequacy of the existing locks in light of their present condition and future needs, and the advisability of their rehabilitation. The scope of this study will focus on the internal structural integrity of the chamber wall monoliths at each lock. The study will review the past history of the St. Lawrence Seaway Development Corporation (SLSDC) structural repairs to these monoliths and will evaluate their current internal condition based on field investigations, detailed finite element analyses and seismic analyses. The Corps will also review previous reports, including the Gannett Fleming reports to identify pertinent information and acceptable existing data for use in the subject evaluation.

The study will be divided into three phases:

Phase I - Field Investigation and Laboratory Testing Program: This phase of the study will determine the current state of the in-situ concrete in the chamber wall monoliths and determine the concrete properties needed for the stress analyses.

Phase II - Static Stress Analysis: The state of stress within the chamber wall monoliths will be examined by performing two-dimensional soil-structure interaction finite element analyses of two monoliths at Eisenhower and one at Snell.

Phase III - Seismic Stress Analysis: Dynamic finite element analyses of two monoliths will be performed to assess their structural integrity during an earthquake. To support the dynamic analysis, a geological-seismological investigation will be conducted to provide the ground motions, time histories, and corresponding response spectra that could be expected during a seismic event.

The findings from these three phases will be used by the Waterways Experiment Station (WES) to assess the adequacy of the existing locks, and for WES to make recommendations on the possible needs for future rehabilitation.

2. **PROJECT MANAGEMENT**

The Buffalo District will be responsible for project management during this evaluation. These duties include the following:

a. Provide monthly progress reports to the SLSDC with copies furnished to North Central Division and Office of the Chief of Engineers.

b. Coordinate the schedule for work being done by various Corps agencies.

c. Initiate funding documents for work by other Corps agencies relating to this evaluation.

d. Monitor funds used and report usage to the SLSDC.

e. Serve as point of contact for SLSDC and all Corps agencies involved.

f. Coordinate OCE and NCD review of all documents generated by this evaluation.

g. Perform other miscellaneous project management tasks as required (example: arrange purchase and delivery of core boxes for drilling)

3. **DETAILED SCOPE OF WORK**

In the following sections, the general scope of work presented in the Intergovernmental Agency Agreement has been expanded to provide the SLSDC the necessary information to assess the adequacy of the proposed study.

<u>Phase I</u> - <u>Field Investigation and Laboratory Testing Program</u>

The field investigation and laboratory testing program will be conducted to provide the necessary parametric data to support the static and seismic analyses. The Mobile District will be conducting the drilling operation. Continuous six inch diameter core will be obtained from vertical core borings will be made six representative locations, four at Eisenhower Lock (in monoliths - N53, N57, S19, and S18) and two at Snell Lock (in monoliths - N56 and S15). Each boring will be approximately 125 feet deep and will penetrate 10-15 feet into the bedrock beneath the locks to check for any unknown rock cavities. A total of 750 linear feet of 6-inch diameter core will be obtained. Core drilling will be performed from the top of the lock walls. For monoliths N53 and N57 at Eisenhower Lock and N56 at Snell Lock, the borings will be located approximately 4 ft from the face of the lock wall to avoid the cable gallery. All borings will be located to insure no interference with the main conduit or any ports for filling and emptying. All coring will be accomplished with a single drill rig and with only one mobilization. Mobile District will

provide a borehole camera to be used to visually inspect the existing conditions of the concrete and rock core holes. WES will have a concrete technologist on site to log and inspect the cores. All core will be wrapped with material to resist drying. The core will be stored on site in space provide by SLSDC.

Three additional core borings will be made depending on the budget and if a distinction can be made between lift joints and cracks. Two will be made at Eisenhower Lock in monoliths S19 and S18 and one in monolith S15 at Snell Lock. The borings will be located as close as possible to the back of the walls. The borings will extend to a depth of approximately 20 feet. These will be used to investigate the possibility of the existent of a crack near the top of the wall where the back of walls changes slope. If the budget permits, these borings will be made in addition to the six primary borings. If not, three of the six primary borings will be reduced in depth to allow these borings to be made.

Once the coring is complete, samples will be selected for laboratory testing. The following tests will be performed.

<u>Concrete Tests</u>

	Type of Test	Number of Tests*
1.	Unconfined Compressive Strength	30
2.	Poisson's Ratio	30
3.	Elastic Modulus	30
4.	Splitting Tensile	30
5.	Specific Gravity, Absorption, Voids	30
6.	Ultrasonic Pulse Velocity	30
7.	Direct Shear	10
8.	Petrographic Examination	2

* These are only approximate numbers of tests and are subject to change after viewing the cores. Extra laboratory tests have been included to compensate for any inconsistent test results due to the 6-inch maximum size aggregate. The results will be presented in a report.

Rock Tests

	Type of Test	Number of Tests*
1.	Unconfined Compressive Strength	12
2.	Poisson's Ratio	12
3.	Elastic Modulus	12
4.	Ultrasonic Pulse Velocity	12
5.	Direct Shear at Interface	2

* These are only approximate numbers of tests and are subject to change after viewing the cores. Extra laboratory tests have been included to compensate for any inconsistent test results due to the 6-inch maximum size aggregate. The results will be presented in a report.

Phase II - Static Stress Analysis

The static stress analysis phase focuses on the evaluation of internal structural integrity of the chamber walls monoliths at Eisenhower and Snell Locks. Of primary concern is the state of stress of the concrete in the chamber walls. The stress conditions in the chamber walls results from gravitational forces on the walls and from external forces applied by the backfill, water, and other possible external loads. The stress analyses will be performed by using the finite element method. Three typical concrete chamber wall monoliths will be analyzed, a north and a south wall at Eisenhower and a south wall at Snell. Each monolith will be analyzed for three loading conditions: normal operation with upper pool in the chamber; normal operation with lower pool in the chamber; and the maintenance condition with the chamber fully dewatered.

The analyses will be performed using the computer program "SOILSTRUCT" written by Dr. G. Wayne Clough and co-workers at Virginia Polytechnic Institute and State University. SOILSTRUCT was specifically developed to model complex soil-structure interaction problems by the finite element method. The code has been used to analyze a wide variety of problems, such as navigation locks, retaining walls, supported excavations, dams, tunnels, foundations, and cofferdams. The analysis will be an incremental soil-structure interaction analysis where the placement of the concrete and the backfill is simulated in order to obtain the induced stresses in the system. The incremental analysis applies the loads to the model in small increments to allow the nonlinear constitutive model for the soil and interface to adjust during the loading increment.

The study will commence by reviewing construction records and previous studies of the locks. From this information, the characterization of the backfill and foundation and a preliminary characterization of the concrete will be made. The chamber wall monoliths will be reviewed for selection of the monoliths for analysis. After selection of the monoliths, a finite element grid will be developed for one of the selected monoliths. The grid will allow for a simulation of a crack from the upper corner of the culvert to exterior surface of the wall and the placement of a post-tension anchor across the crack. The grid will include a sufficient portion of the foundation and backfill as needed to represent their interaction with the structure. Special interface elements will be used between the wall and the foundation and between the wall and the backfill. The rock foundation will be modeled as a linear elastic material with its properties determined from the laboratory testing program. The backfill soil will be represented as a nonlinear material using the Duncan-Chang hyperbolic model. The parameters for the model will be developed from previous soil tests and correlations with published values. The grid will be evaluated for accuracy and adjustments will be made if found necessary. Using the initial characterizations for foundation, backfill, and concrete, a complete analysis of this monolith will be made so that the accuracy of the model can be assessed. Adjustments to the model may also be necessary at this point. Once the model has been proven to be sufficiently accurate, grids for the other selected monoliths will be generated. This work can be done concurrently with the field investigation and laboratory testing.

After the field investigation and laboratory testing has been completed, the final material characterizations will be made for the analyses. Once the final material characterizations have been completed, each of the three monoliths will be analyzed for the three loading conditions. The results from the three analyses will be processed and evaluated in light of the internal structural integrity of the walls. The results will be presented in a report.

Phase III - Seismic Stress Analysis

A seismic analysis will be conducted on either Eisenhower or Snell lock depending upon which structure will be subjected to the highest seismic accelerations. If earthquake-ground accelerations are approximately the same, the seismic analysis will be conducted on Eisenhower Lock, since it has the poorest concrete conditions. To determine the strongest ground motions, a geological-seismological investigation will be conducted to provide the peak values of acceleration, velocity, and duration for comparison of earthquake ground shaking between both sites. Time histories (accelerograms) and corresponding response spectra will be provided to represent site-specific cyclic shaking as it would be felt at the structure experiencing the strongest disturbance.

Once the most critical structure is determined, it will be analyzed considering three loading conditions in conjunction with an operating basis earthquake (OBE). The three load cases are: normal operation with upper pool in the chamber; normal operation with lower pool in the chamber; and the maintenance condition with the chamber fully dewatered. Two different typical lock-wall monoliths will be analyzed using two-dimensional (2-D) finite element procedures with a response spectra from the OBE. These analyses will consider the 2-D dynamic behavior of the structure, hydrodynamic response of the water in the lock, and the dynamic lateral earth pressures. The active dynamic soil pressures will be computed using the Mononobe-Oakbe equation, and the passive pressures will be computed from either a Coulomb or log spiral analysis. The foundation of the lock will be considered rigid. Static stresses from Phase II will be used with the dynamic stresses in order to determine the total state of stress within the structures.

These analyses will provide a preliminary structural evaluation of this structure for a seismic event of the site. If the total state of stress is within the allowable limits of the material, no further seismic stress analyses will be necessary. However, if these analyses indicate that cracking is likely, a seismic analysis using a time-history acceleration record and a nonlinear concrete model should be considered. Such an analysis is beyond the scope of work for this study and would require additional funds.

Mr. Mosher will be the WES point of contact for the study. He will also be the principal investigator for the static analysis. Dr. Hall will be the principal investigator for the seismic analysis. Dr. Krinitzsky will be responsible for conducting the geological-seismological investigation. Dr. Denson will be responsible for overseeing WES personnel performing the logging and inspection of core at the site and directing the concrete testing program.

4. SCHEDULE SUMMARY

The following is a summary of the schedule for the completion of the structural evaluation.

TASK	START DATE	COMPLETE DATE
Notice to Proceed	-----	1/29/90
Initial Site Visit	2/20/90	2/22/90
Prepare Detailed Scope of Work	2/22/90	4/9/90
Field Investigation	5/1/90	10/15/90
Static Analysis	5/1/90	4/15/91
Seismic Analysis	5/1/90	4/15/91

TASK	START DATE	COMPLETE DATE
Draft Report	4/1/91	5/15/91
Review Period	5/15/91	6/15/91
Final Report	5/15/91	7/15/91

5. **FINAL REPORT**

The Buffalo District will be responsible for preparing the final report. The report will summarize all work which took place during the evaluation and review recommendations relating to future maintenance of the locks. The reports generated by WES will be appendices to the main report. The report will be created in draft format for technical review by OCE, NCD, WES, and Buffalo District personnel. Review comments will be addressed and when required will be incorporated into the final report. Following a short Corps review, the final document will be forwarded to SLSDC.

6. **SUMMARY OF COSTS**

Following is a summary of costs necessary to conduct the study:

Drilling, sampling, borehole camera work, and other misc. field work (Mobile District)	$116,000
Site engineer/geologist (WES)	30,000
Laboratory testing program (WES)	40,000
Static analysis (WES)	165,000
Seismic analysis (WES)	152,000
Project management (Buffalo District)	24,000
Final report and coordination (Buffalo District)	34,000
OCE technical assistance	3,000
NCD technical assistance	6,000
TOTAL STUDY COST	$570,000

APPENDIX B: FIELD BORING LOGS

Hole No. N-53

DRILLING LOG	DIVISION	INSTALLATION EISENHOWER LOCK	SHEET 1 OF 11 SHEETS

1. PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT 7 3/4 INCH
2. LOCATION (Coordinates or Station) MONOLITH: N-53 LOCK STATION 328.62	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) US LAKE SURVEY (USLS) - MSL 251.5'
3. DRILLING AGENCY MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL DIAMOND A
4. HOLE NO. (As shown on drawing title and file number) N-53	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 0; UNDISTURBED 0
5. NAME OF DRILLER CARL MOON	14. TOTAL NUMBER CORE BOXES 29
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 0	16. DATE HOLE: STARTED 28 MAY 90; COMPLETED 4 JUN 90
8. DEPTH DRILLED INTO ROCK 10'	17. ELEVATION TOP OF HOLE 251.5
9. TOTAL DEPTH OF HOLE 120.4'	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR M.EILEEN GLYN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
251.5	0		0.0' - 1.3' GOOD CONCRETE MOSTLY SM. AGG. 1 - 3 cm LONG. VERTICAL CRACK ALMOST CONTINUOUS ABUNDANT AIR 1 mm - 1 cm DIA.	56	1	0.0' - 2.5' 1 PC. 1.3'
250	1.3		1.3' - 2.2' GOOD, SOLID CONCRETE. VERTICAL CRACKS CONTINUOUS. ABUNDANT ENTRP. AIR 1 mm - 5 mm DIA.	50		1.4' - 2.5' DRILLED OVER REMAINING CORE IN HOLE.
249	2.2		2.2' - 2.5' RUBBLE	100	2.5	1 PC. 2.4'
248	2.5		2.5' - 4.9' GOOD CONCRETE SOUNDS DULL WHEN HIT, ABUNDANT AIR 1 mm - 5 mm VERTICAL CRACK AT TOP. AGG. MOSTLY SMALL, UP TO 10 cm AVG. 2 cm	100	2	
246	4.9		4.9' - 7.4' GOOD CONCRETE AS ABOVE, EXCEPT FOR VERTICAL HAIRLINE FX. USUALLY AROUND AGG. OCC. THRU AGG. CRACKS HOLD WATER LONGER THAN MASS. T SHAPED STEEL AT 5.5'	84		4.9' - 7.4' 1 PC. 2.0'
245	6.9	MB			6.9	
244	7.4		7.4' - 8.9' GOOD CONCRETE ENTRP. AIR 1 mm - 5 mm DIA. SOUNDS DULL WHEN STRUCK.	87	3	7.4' - 9.9' 1 PC. 2.0'
242	10					9.9' - 17.6'

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PROJECT EISENHOWER '90 | HOLE NO. N-53

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS		Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK		SHEET 2 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			8.9' - 15.0' LT. GRAY, GOOD CONCRETE ENTRP. AIR 1 mm - 5 mm SOUNDS DULL WHEN STRUCK		10.7	9.9' - 17.6' PC. 1.8' 4.3' REC. 6.1' LOSS 2.6' GAIN 1.0'
	11					
	12					
					4	
	13					
	14				14.8	
	15		15.0' - 22.7' GOOD CONCRETE AS ABOVE ENTRP. AIR 0.5 mm - 1 cm DIA. 2 mm AVG. 1 G.			
	16					
					5	
	17					
	18				18.4	17.6' - 25.4' 2 PC. 6.5' 1.3' REC. 7.8' LOSS 2.7' GAIN 2.6'
	19					
	20				6	
	21					
	22					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 3 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
					22.7	
	23		22.7' - 28.5' GOOD CONCRETE, SOUNDS DRUMMY ENTRP. AIR 1 mm - 1 cm DIA. 2 - 3 mm ABUNDANT			
	24				7	
	25					25.4' - 33.8' 1 PC. 5.7' LOSS 3.5' GAIN 2.7'
	26				26.3	
	27					
	28	MB			8	
	29		28.5' - 38.8' GOOD CONCRETE ABUNDANT ENTRP. AIR 1 - 3 mm. MODERATE AMT. OF 1 cm DIA., AIR BUBS USUALLY ADJACENT TO SIDES OF AGG.			
	30				30.4	
	31					
	32				9	
	33					
	34					33.8' - 38.8'

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PROJECT EISENHOWER '90 HOLE NO. N-53

RN039-048 SYS 2 graphics by VISUAL PRODUCTS SECTION - ITL for INFORMATION TECHNOLOGY LABORATORY - WES

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 4 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	35 – 38	MB			34.6 10	33.8' - 38.8' 2 PC. 7.6' 2.4' REC. 10.0' LOSS 0.0' GAIN 5.0'
	39 – 42	MB	38.8' - 43.0' GOOD CONCRETE AIR 1 mm DIA. THRU OUT OCC. 1 cm DIA.	25	39.4 11 43.7	38.8' - 46.2' 2 PC. 0.6' 3.6' REC. 4.2' LOSS GAIN 0.0'
	43 – 46	MB MB	43.0' - 52.6' GOOD CONCRETE ABUNDANT AIR 1 mm DIA. - 2.5 cm IRREGULAR SHAPE AVG. 1 - 2 mm SOUNDS DRUMMY		12	43.0' - 52.6' BEATEN OUT OF CORE BARREL BIT STUCK ON TIGHT DON'T USE FOR TESTING. WELL BROKEN.

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 5 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	47			175	12	46.2' - 52.5' 8 PC. 0.7' 0.7' 1.0' 1.0' 1.2' 1.4' 1.8' 0.7' 0.4' CHIPS 9.5' TOTAL GAIN 3.6'
	48	MB			48.2	
	49					
	50				13	
	51					
	52		BROKEN 52.0' - 52.5'		52.5	
	53		52.5' - 61.1' GOOD CONCRETE ENTRP. AIR 1 mm - 1.5 cm DIA., AVG. 1 - 3 mm DIA. SOUNDS DULL WHEN STRUCK. GOOD AGG. DIST.	100	14	52.5' - 61.1' 7 PC. 4.5' 1.3' 0.6' 0.4' 0.6' 0.5' 0.7' BROKEN 8.6' DIFFICULT TIME RETRIEVING CORE.
	54					
	55					
	56					
	57				57.0	BOTTOM OF RUN BROKEN
	58					

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MAR 71

PROJECT EISENHOWER '90 | HOLE NO. N-53

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 251.5 USLS | Hole No. N-53

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: EISENHOWER LOCK | SHEET 6 OF 11 SHEETS

ELEVATION (a)	DEPTH (b)	CLASSIFICATION OF MATERIALS (Description) (d)	% CORE RECOVERY (e)	BOX OR SAMPLE NO. (f)	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) (g)
	59–61	VERTICAL BREAK 60.4' - 61.1'		15 / 61.1	
	61–66	61.1' - 66.1' GOOD CONCRETE ENTRP. AIR 1 mm - 5 mm DIA., AVG. 2 mm, SLIGHTLY LESS AIR THAN ABOVE. 1 LG. PC. AGG. 5" DIA AT 63.4' REMAINS: GOOD DIST.	100	16 / 66.1	61.1' - 71.1' 7 PC. 0.7' 1.2' 1.3' 1.8' RUN FELL OUT OF BARREL ON TO GROUND.
	66–70	66.1' - 71.4' GOOD CONCRETE ENTRP. AIR SAME AS ABOVE		17	

Depth scale: 59, 60, 61, 62, 63, 64, 65, 66, 67, 68, 69, 70

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PROJECT: EISENHOWER '90 | HOLE NO.: N-53

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53

PROJECT	INSTALLATION	SHEET
EVALUATION OF EISENHOWER AND SNELL LOCKS	EISENHOWER LOCK	7 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
					70.6	
	71		BROKEN AT 71.0'			
			71.1' - 76.5' GOOD CONCRETE ENTRP. AIR 1 mm - 1 cm DIA. OCC. 2 cm DIA. AIR AVG. SIZE OF AGG. INCREASED TO 2 - 3 IN. FREQ. LG. PC. AGG. UP TO 6"			71.1' - 81.0' 2 PC.
	72					REDO MB RUN 71.1' - 81.0'
	73				18	
		MB				
	74					DRILLER 80.7' - 70.9'
	75	MB			75.3	
	76	MB		100		
			76.5' - 81.0' POOR CONCRETE			4 PC. 2.8' 2.6' 3.6' 0.9'
	77		POOR BONDING W/ AGG. POSSIBLE MICRO FRACTURING THROUGHOUT CEMENT.		19	9.9' TOTAL
	78	MB				
	79	MB			79.5	
	80	MB				
		MB			20	
	81	?	81.0' - 81.7' MOD. GOOD CONCRETE			81.0' - 91.0'
		LIFT	81.7' - 86.5' POOR CONCRETE. LOOKS OLD, POOR BONDING W/AGG.			
	82					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS		Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK		SHEET 8 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			AGG. SIZE AVG. DEC. TO 2.5 cm SQ. 81.5' - 86.5' SMALL. RED PARTICLES LOOKS LIKE BRICK IN CEMENT. CRACKS THROUGH OUT. LONGEST PC. REC. 1.0'			
	83				83.3	
	84		ABUNDANT ENTRP. AIR 1 mm DIA. AVG.			
	85				21	
	86			100		
			86.5' - 91.0' GOOD CONCRETE AVG. AGG. SIZE INCRE. TO 4.0 cm BETTER BONDING W/ AGG. THAN ABOVE. LESS ENTRP. AIR DIA. = 1 mm - 1 cm AT CRACK AT 86.5'			
	87				87.3	
	88					
	89				22	
	90					
					91.0	
	91		91.0' - 95.9' GOOD CONCRETE ENTRP. AIR 2 mm - 5 mm OCC. IRREG. BUB 1 - 2 cm LONG. GOOD CONSOLIDATION GOOD AGG. DIST.			91.0' - 101.0'
	92			50	23	
	93					
	94					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 9 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	95–101.1		95.9' - 101.1' GOOD CONCRETE ENTRP. AIR 1 mm - 5 mm		95.9 / 24 / 100.2	
	101.1–102		101.1' - 102.0' CEMENT DEGRADED PITTED.	140	25	101.0' - 105.4' 8 PC. 0.7' 1.3' 0.4' 1.5' 0.4' 0.6' 0.3' 0.9' REDO 95.9' - 102.0'
	102–104.8		102.0' - 104.8' MOD. GOOD CONCRETE. CEMENT DEGRADED PITTED THRU OUT HIGH CONCENTRATION OF SMALLER AGG. ARE SIZE 3 cm LONG.		104.8	
	104.8–106		104.8' - 110.4' MOD. GOOD CONCRETE. SAME AS ABOVE, SOUNDS V. DRUMMY.		26	105.4' - 111.9' 1 PC. 2.8'

Depth scale: 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 10 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
						105.4' - 111.7' WENT BACK OVER CORE LEFT IN HOLE AFTER MOD. SPRING
	107					2 PC. REC. 5.6' 1.6'
	108	MB			108.6	
	109			95		
	110	MB			27	
	111		CONCRETE/ROCK INTERFACE NO BOND 110.4' - 112.0' MISSING SIDE OF JT.: DOLOMITE V. HARD. BLACK/GRAY V. FINE LAMINATED			
	112	MB	112.0' - 120.4' V. GOOD V. HARD, BLACK/GRAY LAMINATED DOLO. INTRBD. W/SHALE. CLAY LAYER .03' THICK AT 113.4' SHALE. LAYERS AT ALL BREAKS .05' - 0.2' THICK.		112.0	112.0' - 120.4' 6 PC. 1.6' 0.05' MS 1.3' 0.05' MS 1.7' 0.05' MS 1.1' 0.2' 0.05' MS 1.1' 0.2' CHIPS
	113	MB				
	114				28	ALSO REC. 1.2 OF LEFT SIDE OF VERTICAL FX. AT 110.4' - 112.0'
	115	MB				
	116	MB			116.6	
	117	MB	116.6' - 119.4' MOTTLED DOLO. CAL. BLEBS UP TO 4 cm LONG, 3 mm THICK		29	
	118	MB				

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DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. N-53
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 11 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	119	MB				
			119.2' - 119.4' SHALE BROKEN			
	120					
						120.4' - BOTTOM OF HOLE. DID NOT RECOVER LAST 1.0' - BAREL WAS FULL. - BROKE SAMPLE IN HOLE AT SHALE BED/ DOLOMITE INTERFACE - DIFFICULT TIME RETRIVING CORE. FROM THIS HOLE, FROM WED 30 JUNE TIL FINISH 4 JUNE.

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PROJECT EISENHOWER '90 | HOLE NO. N-53

Hole No. N-57

DRILLING LOG	DIVISION	INSTALLATION EISENHOWER LOCK	SHEET 1 OF 11 SHEETS

1. PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT 7¾" DIAMOND
2. LOCATION (Coordinates or Station) MONOLITH: N-57 LOCK STATION 506.00	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) USLS - MSL
3. DRILLING AGENCY MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 1500
4. HOLE NO. (As shown on drawing title and file number) N-57	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 0; UNDISTURBED 0
5. NAME OF DRILLER CARL MOON	14. TOTAL NUMBER CORE BOXES 30
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ______ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN 0	16. DATE HOLE: STARTED 5 JUNE 90; COMPLETED 8 JUNE 90
8. DEPTH DRILLED INTO ROCK 12.4	17. ELEVATION TOP OF HOLE 251.5'
9. TOTAL DEPTH OF HOLE 125.0	18. TOTAL CORE RECOVERY FOR BORING %
	19. SIGNATURE OF INSPECTOR M.EILEEN GLYNN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
251.5	0		0.0' - 3.0' GOOD NEW CONCRETE, ENTRP. AIR 1 mm - 1 cm DIA. MOSTLY V. SMALL AIR BUBS. 0.4' - 0.8' MISSING BECAUSE IT WAS BEATEN + BROKEN OUT OF CORE BARREL CATCHER.	100		0.0' - 0.8' 1 PC. 0.4' RUBBLE 0.4'
	1			100	1	0.8' - 3.0' 1 PC. 1.2'
	2	MB				
	3	MB	3.0' - 4.7' GOOD NEW CONCRETE,	80	4.7	3.0' - 5.4' 1 PC. 1.9'
	5		4.7' - 7.1' GOOD NEW CONCRETE, GOOD CONSOL. WHITE RXN. MTL. ON BREAK 4.7' AIR ENTRP. 1 mm - 5 mm DIA. AVG. 1 mm		2	5.4' - 7.1' REC. 2 PC. 1.8 4.6
	6	MB				
	7	MB	7.1' - 8.2' GOOD CONCRETE, PC. OF STEEL T-SHAPE AT 7.2' - AIR 1 mm - 2 mm DIA.		8.2	7.1' - 9.4' HAVING TROUBLE LIFTING SAMPLE 1 ST. TRY REC. 0.7' 2 ND. TRY 0.5' QUIT
	8	MB				
	9				3	
241.5	10					

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PROJECT EISENHOWER '90 | HOLE NO. N-57

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 215.5	Hole No. N-57
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 2 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB				
	11		8.2' - 17.3' GOOD CONCRETE, WELL CONSOLIDATED GOOD AGG. DIST. ENTRP. AIR 1 mm - 5 mm DIA. NOT ABUNDANT, OCC. LG. BUB 2 cm LONG.		3	94.0' - 17.3' 4 PC. 2.0' 5.7' 0.1' CHIPS 1.1' 0.3' 9.2'
		MB			11.8	
	12					
	13					
	14			114	4	
	15					
		MB			15.8	
	16					BOTTOM 0.3' WAS BEATEN OUT OF CORE BARREL, DON'T TEST 15.9' - 17.3'
	17	MB				
		MB	17.3' - 26.0' GOOD CONCRETE, ENTRP. AIR : MOSTLY SMALL. 1 mm - 2 mm, OCC. 1 cm - 2 cm DIA.		5	17.3' - 26.0' 3 PC. 4.2' 1.4' 3.0'
	18					
	19			100		
		MB			19.9	
231.5	20					
	21				6	
		MB				
	22					

ENG FORM 1836-A PREVIOUS EDITIONS ARE OBSOLETE.
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PROJECT EISENHOWER '90 | HOLE NO. N-57

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 215.5 | Hole No. N-57

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: EISENHOWER LOCK | SHEET 3 OF 11 SHEETS

ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant)
		MB				
	23		— MB AT V. LG. BOULDER 23.0' - 26.0' LG. ENTRP. AIR MORE FREQ.		6	
	24					
		MB			24.8	
	25					
		MB				
	26		26.0' - 35.5' GOOD CONCRETE, GOOD CONSOLIDATION.			26.0 - 35.5' 2 PC. 1.0' 8.5'
	27		— BREAK AT LG. BOULDER POSIBLE LIFT LINE ? (27.0')		7	
	28					
	29	MB			29.3	
			ENTRP. AIR 1 mm - 5 mm OCC. ELONGATED BUBS AT 29.6'			
221.5	30			100		
	31				8	
	32	MB				
	33					
	34					

ENG FORM 1836-A PREVIOUS EDITIONS ARE OBSOLETE.
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PROJECT: EISENHOWER '90 | HOLE NO. N-57

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. N-57
PROJECT EISENHOWER LOCK	INSTALLATION	SHEET 4 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
						34.1
	35	MB				
		MB				
	35.5		35.5'-45.5' GOOD CONCRETE. AGG. MOSTLY SMALL 1 cm-4 cm AVE. 2 cm ENTRP. AIR 1 mm-1 cm AVE. NOT ABUNDANT.			35.5'-45.5' 5 PC. 2.50' 3.00' 2.60' 1.30' 0.60' 10.00'
	36				9	
	37					
	38	MB			38.0	
	39					
211.5'	40			100	10	
	41	MB				
	42	MB			42.3	
	43	MB				
	44				11	
	45	LIFT?				
	45.5	MB				45.5'-55.0'
	46					

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PROJECT	HOLE NO.
EISENHOWER '90	N-57

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 251.5 | Hole No. N-57

PROJECT: EISENHOWER LOCK | INSTALLATION: | SHEET 5 OF 11 SHEETS

ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant)
			45.5'-52.8' GOOD CONCRETE. ENTRP. AIR 1 mm-1 cm THROUGHOUT.			45.5'-55.0' 2 PC. 4.50' 5.00'
	47					
	48				12	
	49					
201.5'	50	LIFT JT. ?	BREAK AT 50.0' IS PLANAR BUT A LITTLE ROUGH. 50.0'-52.8' - MINOR DEGRADATION AROUND AGG. NOT SEEN ABOVE OR BELOW.		50.0	
	51			100		
	52	MB			13	
	53		52.8'-55.0' GOOD CONCRETE. ENTRP. AIR 1 m-3 mm DIA THROUGHOUT.			
	54		54.8'-55.0' BROKEN AND BEATEN OUT OF BARREL.		54.8	
	55	LIFT?	55.0'-65.0' GOOD CONCRETE. ENTRP. AIR 1 mm-1 cm DIA OCC. 2 cm IRREG. BUB.			55.0'-65.0' 3 PC. 6.40' 1.90' 1.70' BOTTOM OF RUN WAS BEATEN W/HAMMER TO LOOSEN. DO NOT TEST 63.5'-65.0'.
	56				14	
	57					
	58					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. N-57
PROJECT EISENHOWER LOCK	INSTALLATION	SHEET 6 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	59	COLD JT?	58.8'-61.4' LESS ABUNDANT ENTRP. AIR 1 mm-2 mm, OCC. 1 cm DIA.		58.8	
191.5'	60					
	61	LIFT JT?		100	15	
	62					
	63	MB			63.3	
	64	MB				
	65	MB	65.0'-75.0' GOOD CONCRETE. ENTRP. AIR 1 mm-5 mm DIA. OCC. 1 cm DIA., GOOD CONSOLIDATION.		16	65.0'-75.0' 5 PC. 2.80' 2.20' 1.20' 2.00' 1.50' 0.30' CHIPS
	66					
	67	MB		100	67.8	
	68					
	69				17	
181.5'	70	LIFT?				

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PROJECT EISENHOWER '90 | HOLE NO. N-57

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 | Hole No. N-57

PROJECT EISENHOWER LOCK | INSTALLATION | SHEET 7 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	71	MB			71.2	
	72					
				100		
	73	MB			18	
						DO NOT TEST 73.2'-75.0'
	74					
			74.7'-75.0' BROKEN		75.0	
	75	LIFT?	75.0'-85.0' GOOD CONCRETE. ENTRP. AIR 1 mm-5 mm THRU OUT, OCC. 2 cm LONG IRREG. BUB.			75.0'-85.0' 4.20' 4.20' 1.20' 0.40' 10.00'
	76					
	77				19	
	78					
				100		
	79	MB			79.2	
161.5'	80		TWO PARALLEL VERTICAL CRACKS 80.0'-81.8'.			
					20	
	81					
	82					

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 251.5

Hole No. N-57

PROJECT: EISENHOWER LOCK

INSTALLATION:

SHEET 8 OF 11 SHEETS

ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant)
			82.3'-82.7' ROUND GROUTED HOLE. BROWN CEMENT. V. SANDY ONLY ON ONE SIDE.			
	83	MB				
	84		CRACK AT 84.0'			
		MB	STEEL BAR 1.5 cm DIA. ONE SIDE ONLY			
	85	LIFT?	85.0'-95.0' GOOD CONCRETE. GOOD CONSOLIDATION ENTRP. AIR 1 mm-3 mm DIA. OCC. 1-2 cm LONG IRREG. BUB.		21	85.0'-95.0' 1.90' 5.90' 1.00' 0.40' CHIP 0.30' CHIP 0.40' 0.10' CHIP
	86	MB				
	87					
	88					
	89			100		BOTTOM BEATEN OUT OF CARE BARREL. DO NOT TEST LAST 22' OF RUN.
151.5'	90				22	
	91					
	92	MB				
	93					
	94					

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PROJECT: EISENHOWER '90

HOLE NO. N-57

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 | Hole No. N-57

PROJECT EISENHOWER LOCK | INSTALLATION | SHEET 9 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB				
		MB				
	95	MB	95.0'-103.0' GOOD CONCRETE. GOOD CONSOLIDATION, GOOD AGG. DIST. ENTRP. AIR 1 mm-5 mm DIA., OCC. 1 cm DIA.		23	95.0'-105.0' 4 PC. 7.00' 1.80' 0.80' 0.40'
	96					
	97	MB	MINOR DEGRADATION AROUND AGG. 101.5'-103.0'.		97.0	
	98	MB				
	99				24	
141.5'	100					
	101					
	102	MB	BREAK AT LG BOULDER 102.0'		102.0	
	103		103.0'-105.0' POOR CONCRETE. LG. VOIDS. STRONG.			
	104				25	
	105		105.0'-105.4' POOR CONCRETE. STRONG, BUT LG. VOIDS.			105.0'-115.0' 2 PC. POKER CHIP BROKEN
	106					

ENG FORM MAR 71 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT EISENHOWER '90 | HOLE NO. N-57

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 251.5 | Hole No. N-57

PROJECT: EISENHOWER LOCK | INSTALLATION: | SHEET 10 OF 11 SHEETS

ELEVATION (a)	DEPTH (b)	LEGEND (c)	CLASSIFICATION OF MATERIALS (Description) (d)	% CORE RECOVERY (e)	BOX OR SAMPLE NO. (f)	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) (g)
	107, 108, 109	MB, MB	105.7'-112.6' GOOD CONCRETE. GOOD CONSOLIDATION ENTRP. AIR 2 mm-5 mm DIA. OCC. 1 cm DIA. AIR LESS ABUNDANT AT 111.0'-112.6'.		26	105.0'-115.0' 2 PC. 3.70' 4.70' CHIPS 1.60'
131.1'	110, 111	MB		100	111.3	
	112, 113	MB	112.6'-113.3' GOOD DOLOMITE. HARD. GRAY.		27	
	114		113.3'-115.0' POOR ROCK. DOLOMITE INTERBEDED W/SHALE. EVERY 0.1' TO 0.4'. MOSTLY BROKEN - SOME POKER CHIP RECOVERY.			
	115		NOT ONE PC. INTACT 113.3'-115.0'.		115.0	
	116		115.0'-115.3' SHALE WEAK. BLACK POKER CHIP 115.3'-117.1' DOLOMITE V. HARD.		28	115.0'-125.0' 16 PC. VERTICAL BREAK AT 115.3'-116.2'
	117, 118		117.1'-119.0' DOLOMITE INTERBED W/SHALE BEDS AND STRINGERS.			

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 | Hole No. N-57

PROJECT: EISENHOWER LOCK | INSTALLATION: | SHEET 11 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			MB			
	119		119.0'-125.0' DOLOMITE V. HARD LT. GRAY (DRY) BLACK WET. OCC. SHALE STRINGER + BEDS.			
	120		MB			
			MB		29	
	121		MB			
				100		
	122		MB MB MB			
					122.8	
	123		MB			
			MB			
			MB			
	124		MB		30	
	125		MB		125.0	BOTTOM OF HOLE.
	126					
	127					
	128					
	129					
	130					

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PROJECT EISENHOWER '90 | HOLE NO. N-57

Hole No. S-17

DRILLING LOG	DIVISION	INSTALLATION EISENHOWER LOCK	SHEET 1 OF 11 SHEETS

1. PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT 6 INCH DIAMOND
2. LOCATION (Coordinates or Station) MONOLITH: S-17 LOCK STATION 192.50	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) US LAKE SURVEY (USLS) 251.5'
3. DRILLING AGENCY MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 1500
4. HOLE NO. (As shown on drawing title and file number) S-17	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 0; UNDISTURBED 0
5. NAME OF DRILLER CARL MOON	14. TOTAL NUMBER CORE BOXES 30
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 14 MAY 90; COMPLETED 19 MAY 90
7. THICKNESS OF OVERBURDEN 0	17. ELEVATION TOP OF HOLE 251.5
8. DEPTH DRILLED INTO ROCK 13.2'	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 122.8'	19. SIGNATURE OF INSPECTOR M.EILEEN GLYN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
251.5	0–2.5	NB, MB	0.0' - 2.5' GOOD CONCRETE NB V. WEATHERED SURFACE W/IRON STAINING. + ENTRAIN AIR 1 mm DIA. 0 cc. ENTRP. AIR THRU OUT MB HAS ENTRAIN BUB ≈ 1 mm DIA. THRU OUT	100	1	DRILL TIME ≈ 20 MIN. HYD. PRES ≈ 300 RUN 1 0.0' - 2.5'
	2.5–5.0	MB, MB	2.5' - 5.0' GOOD CONCRETE AGG. EVEN DIST. 2.5' - 3.5' LARGER ENTRP. AIR 0.5 cm - 2 cm LONG 3.5' - 5.0' ENTRP. AIR LESS FREQ. (3-5 mm DIA.)	100	3.8	DRILL TIME ≈ 20 MIN. HYD. PRES ≈ 300 RUN 2 2.5' - 5.0'
245.5	5.0–7.2 (6)	MB	5.0' - 7.2' GOOD CONCRETE HARD. AIR ENTRP. (1 mm-1 cm) MB AT LG. AGG.	100	2 7.2	DRILL TIME ≈ 20 MIN. HYD. PRES ≈ 300 RUN 3 5.0' - 7.2'
	7.2–9.7	MB	7.2' - 9.7' GOOD CONCRETE HARD. AIR ENTRP. + ENTRAIN. ALONG MB., BREAK AT BOULDER. + WH. CRUST MTL. ON BRK.	100	3	DRILL TIME ≈ 20 MIN. HYD. PRES ≈ 300 RUN 4 7.2' - 9.7'
241.5	10		SEE NEXT SHEET (2)			

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT EISENHOWER LOCK HOLE NO. S-17

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 USLS | Hole No. S-17

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: EISENHOWER LOCK | SHEET 2 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	9.7–11.7		9.7' - 14.7' GOOD CONCRETE ENTRP. AIR ≈ 1 cm DIA. THRU OUT 1 LG. BUB AT 10.7' (2 cm DEEP X 2 cm LONG X 1 cm WIDE)	100	3	HYD. PRES ≈ 300 DRILL ACTION : SMOOTH 1 PC. RECOVERED 5.0' RUN 5 9.7' - 14.7'
	11.7	MB	ENTRAIN AIR ≈ 1-3 mm DIA. THRU OUT AGG. GOOD DISTR.		11.7	
	12.9	MB				
	11.7–14.7				4	
	14.7	MB	14.7' - 18.7' GOOD CONCRETE ABUNDANT ENTRP. AIR ≈ 1 - 2 cm DIA. BREAK AT AGG NOT ACROSS AGG.			14.7' - 24.3' RUN 6
	16.2		ENTRAIN AIR ≈ 1-2 mm DIA.		16.2	
	16.2–18.7			100	5	
	18.7	MB	18.7' - 24.3' GOOD CONCRETE LESS ENTRP. AIR			
	20.7				20.7	
	22					

ENG FORM 1836-A PREVIOUS EDITIONS ARE OBSOLETE. MAR 71

Hole No. S-17

DRILLING LOG	DIVISION	INSTALLATION EISENHOWER LOCK	SHEET 3 OF 11 SHEETS

1. PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT 6 INCH DIAMOND
2. LOCATION (Coordinates or Station) MONOLITH: S-17 LOCK STATION 192.50	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) US LAKE SURVEY (USLS) 251.5'
3. DRILLING AGENCY MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 1500
4. HOLE NO. (As shown on drawing title and file number) S-17	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED 0 — UNDISTURBED 0
5. NAME OF DRILLER CARL MOON	14. TOTAL NUMBER CORE BOXES 30
6. DIRECTION OF HOLE ☐ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE — STARTED 14 MAY 90 — COMPLETED 19 MAY 90
7. THICKNESS OF OVERBURDEN 0	17. ELEVATION TOP OF HOLE 251.5
8. DEPTH DRILLED INTO ROCK 13.2'	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE 122.8'	19. SIGNATURE OF INSPECTOR M.EILEEN GLYN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	23					
					6	
	24					
			24.3' - 27.4' GOOD CONCRETE HARD, ENTRAIN AIR 1 mm DIA. ENTRP. AIR = 3 mm DIA.- 2 cm LONG LARGE VOID AT 25.7', POSSIBLE COLD JT. AT 25.2' NO COLOR CHANGE JUST A THIN WHITE LINE, COULD BE A HAIRLINE CRACK.			DRILL TIME = 45 MIN. HYD. PRESS 300 RUN 7 24.3' - 34.3' 1 PC. 3.1'
	25	MB			25.1	
	26			30		
	27	MB			7	
			27.4' - 34.3' ENTRP. AIR 5 mm - 1 cm DIA., ENTRAIN AIR = 1 - 2 mm			
	28					
	29	MB			29.4	
	30					
	31				8	
			CRACK AT 31.5'			
	32					

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PROJECT EISENHOWER LOCK — HOLE NO. S-17

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DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. S-17
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 4 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	33 – 34.3				33.1	
	34.3 – 37.3		CRUMBLY AT 34.3' - 34.8' CORE INTACT EXCEPT FOR SIDE WEDGE. 34.8' - 37.3' GOOD CONCRETE	170	9	DRILL TIME 15 MIN. 2 PC. 3.1' 7.0' RUN 8 34.3' - 37.3'
	37.3 – 42		37.3' - 40.0' GOOD CONCRETE AIR 1 mm - 1 cm DIA.	100	37.3	DRILL TIME 50 MIN. 4 PC. 2.6' 2.8' 3.1' 0.4' RUN 9 37.3' - 46.2'

Depth scale: 33, 34, 35, 36, 37, 38, 39, 40, 41, 42

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. S-17
PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION: EISENHOWER LOCK	SHEET 5 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB			42.3	
		MB				
		MB				
	43					
	44					
	45					
		MB	BREAK ALONG LG. AGG.		45.8	
	46	MB				
			46.2' - 49.8' GOOD CONCRETE 0.2 - 1.0 cm DIA. AIR ENTRP. BUBS SEEM TO BE LOCATED ADJACENT TO AGG.			DRILL TIME 50 MIN.
	47					
	48					
	49					
		MB				
	50		49.8' - 52.7' AIR ENTRP. BUB SEEM TO BE DEEPER ≈ 1cm INTO CORE. W/ 1 cm DIA.			
	51					
		MB				
	52					
		MB				
	53		52.7' - 55.2' GOOD CONCRETE LESS ENTRP. AIR EXCEPT ONE LG. BUB AT 54.7', 2.5" LONG 1 cm DEEP			
	54					

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PROJECT	HOLE NO.
EISENHOWER LOCK	S-17

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. S-17
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 6 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	55	MB				
		MB	55.2' - 55.9', 3 LG BUBS 1.5 cm DIA. X 1 cm DEEP			
	56		55.9' - 59.8' GOOD CONCRETE AIR ENTRP. 1 cm DIA.		14	DRILL TIME 70 MIN. 3 PC. 3.9' 2.4' 3.5'
	57					
	58	MB			58.4	
	59	MB				
	60		59.8' - 62.2' AIR ENTRP. AVG. 0.5 cm DIA.		15	
	61					
	62	MB	62.2' - 63.7' LESS ENTRP. AIR (1 mm - 5 mm DIA.)			
	63	MB	63.7' - 65.7' MORE ENTRP. AIR, AVG. 0.5 cm 2 LG. BUBS 1.5 cm LONG		63.1	
	64				16	
	65	MB				
	66					

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PROJECT EISENHOWER '90 | HOLE NO. S-17

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. S-17
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 7 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			65.7' - 69.6' GOOD CONCRETE AIR ENTRP. AVG. 2 mm DIA. THRU OUT. SOME LG. AT 1 cm DIA.			DRILL TIME REC. 5 PC. 3.9' 1.5' 2.8' 0.8' 0.7'
	67					
		MB			67.5	RUN: 65.7' - 75.6'
	68					
			REBAR 68.7'			
	69					
		MB			17	
	70					
	71	MB	REBAR 71.0'			
			71.1' - 74.0' GOOD CONCRETE AIR ENTRP. AVE 2 - 5 mm DIA.			
		MB			71.9	
	72					
	73					
		MB	REBAR AT 73.9' 73.9' - 75.6' GOOD CONCRETE LESS AIR ENTRP. EXCEPT 1 LG. BUB AT 75.3' ≈ 1.5 cm LONG		18	
	74					
		MB				
	75					
		MB			75.6	
			75.8' - PC. OF REBAR			75.6' - 85.6' 2.3' 0.2' CHIPS 1.6' 1.45' 3.5' 0.8'
	76					
	77				19	
	78					

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PROJECT EISENHOWER '90 HOLE NO. S-17

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 USLS | Hole No. S-17

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: EISENHOWER LOCK | SHEET 8 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			CHIP AT 77.9' 78.2' - 79.7' GOOD CONCRETE			MISSING CHIPS AT 78.1' AND 84.7'
			REBAR AT 78.7' RUSTED		19	
	79		AIR ENTRP. 2 - 3 mm DIA. ABUND. AT 79.2' - 79.7'			
	80	?	79.7' - 81.15' GOOD CONCRETE PC. OF AGG. (4 IN) AIR ENTRP. W/ RCT MTL. AT BREAK POSSIBLY WEATHERED. REBAR AT 80.1'			
	81	MB	81.15' - 84.6' GOOD CONCRETE LESS AIR ENTRP. 3 mm DIA.		20	
	82					
	83	MB	REBAR AT 83.2' CRACK IN AGG. NEAR REBAR	98.5		
	84				83.2	
	85		84.7' - 85.6' GOOD CONCRETE INC. IN AIR ENTRP. THROUGHOUT 1 BUB AT 1 cm DIA.		21	DO NOT USE LAST PC. FOR TESTING (WAS BEATEN OUT OF CORE BARREL)
	86		85.6' - 91.6' GOOD CONCRETE AIR ENTRP. 1 cm DIA. + 1 cm DEEP -1			DRILL TIME = 70 MIN.
	87	MB	87.0' - 91.6' GOOD CONCRETE AIR ENTRP. AVE. 1-3 mm OCC. 1 cm DIA. AIR BUB ADJ. TO AGG.	100	87.0	85.6' - 95.1' 4 PC. 6.00' 2.25' 1.10' 0.40' 0.15'
	88					
	89	MB			22	
	90					

ENG FORM 1836-A PREVIOUS EDITIONS ARE OBSOLETE.
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PROJECT: EISENHOWER '90 | HOLE NO: S-17

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. S-17
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 9 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	91	MB			22	
					91.6	
	92		91.6' - 94.95' GOOD CONCRETE AIR ENTRP. 91.6' - 92.0' 1 cm DIA.	100		
	93				23	
	94	MB	AIR ENTRP 93.6' - 94.5' 1 - 2 cm DIA.			
			94.95' - 95.1' BROKEN		95.1	
	95		95.1' - 99.7' GOOD CONCRETE ENTRP. AIR THROUGH OUT AVG. DIA. 1 cm			95.1' - 105.2'
	96					
	97			100	24	
	98					
	99	MB			99.7	
	100		99.7' - 101.8' GOOD CONCRETE OCC. LG. AIR BUBS 1 - 3 cm LONG			
	101	MB				
	102					

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PROJECT	HOLE NO.
EISENHOWER '90	S-17

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 USLS | Hole No. S-17

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: EISENHOWER LOCK | SHEET 10 OF 11 SHEETS

ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant)
		MB	101.8' - 105.1' GOOD CONCRETE ENTRP. AIR 0.5 cm DIA.			
	103	MB			103.8	
	104			100		
			CHIPS AT 104.6' - 104.8'			
	105	MB				
	106		105.2' - 109.6' GOOD CONCRETE SOUNDS DRUMMY, LOOKS MORE POROUS THAN ABOVE HIGH CONCENTRATION OF ENTRP. AIR 3 mm - 2 cm LONG AVG. 3 mm DIA.		26	105.2' - 115.4' 6 PC. 5.5' 0.2' CHIPS 0.7' 1.7' 0.4' 0.3' CHIPS 0.55' 0.5' DRILL TIME 80 MIN.
	107			96.6		
	108				108.0	
	109	BOND				
	110		109.6' ROCK / CONCRETE CONTACT, GOOD BOND V. STRONG ROCK. DK GRAY DOLOMITE TO 115.4'		27	
		JT	JT.			
	111	MB	CRACK AT CONTACT IN ROCK, PROBABLY MECH.		111.6	
	112					
	113	MB				
	114		CHIPS AND DISCS 113.7' - 114.0'			

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USLS	Hole No. S-17
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 11 OF 11 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			ELONGATED HOLES AT 114.2'			
		JT	WEATHERED JT. 114.9'		28	
	115					
			115.4' - 118.2' GOOD HARD, BLACK / GRAY LAMINATED DOLOMITE			
	116				116.2	
		JT	VERTICAL JT. AT 116.2'			
	117					
		JT	CLAY / SHALE STRINGER			
	118	JT			29	
			118.2' - 120.0' GOOD HARD DOLO. MOTTLED DK + LT. GRAY, CALCITE BLEBS UP TO 4 cm LONG			
	119	MB		100		
			CLAY / SHALE STRINGER			
	120		120.0' - 122.8' GOOD HARD, GRAY DOLOMITE SHALE BEDS UP TO 0.2' THICK AT 120.4'		120.4	
	121					
			STYOLITE		30	
	122		POSSIBLE JT.			
		MB				BOTTOM OF HOLE
	123					

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PROJECT EISENHOWER '90 | HOLE NO. S-17

Hole No. S-19

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 12 SHEETS

1. PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS
2. LOCATION (Coordinates or Station): 275.00'-7.825' FROM LOCK CHAMBER
3. DRILLING AGENCY: C O E MOBILE DISTRICT
4. HOLE NO. (As shown on drawing title and file number): S-19
5. NAME OF DRILLER: CARL MOON
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 0
8. DEPTH DRILLED INTO ROCK: 129.0'-110.7'-18.3'
9. TOTAL DEPTH OF HOLE: 129.0'
10. SIZE AND TYPE OF BIT: 7-3/4" DIAMOND
11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL-USGLS
12. MANUFACTURER'S DESIGNATION OF DRILL: DIAMOND
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 0; UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES: 30
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 21 MAY 90; COMPLETED 25 MAY 90
17. ELEVATION TOP OF HOLE: 251.5'
18. TOTAL CORE RECOVERY FOR BORING: 128.7' %
19. SIGNATURE OF INSPECTOR: M. EILEEN GLYNN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
251.5'	0		0.0'-2.5'- GOOD CONCRETE. AVE. SIZE AGG. 2.5 cm LONG. AIR ENTRP. 1 mm-5 mm DIA. THRU OUT.	100	1	DRILL TIME ≈ 20 MIN. 1 PC. INTACT 2.5' PRESS ≈ 275 PSI
250.0'	1		1.7'-2.5' - AGG. SIZE AVE. INCREASES TO 3 IN.			
	2.5		2.5'-5.0' - GOOD CONCRETE. AIR ENTRP. 3 mm-1 cm LONG, MORE ABUNDANT AT 4.5'-5.0', AVE. AGG 3 IN. DIA.	100		DRILL TIME ≈ 20 MIN. PRESS ≈ 275 PSI 1 PC. 2.5' INTACT
	4		AVE. AGG. 2 cm DIA. 4.5'-5.0'- SOUNDS DRUMMY			
	5		5.0'-7.2' - GOOD CONCRETE. AIR ENTRP. 3 mm-2 cm LONG AGG. DIST IS LG. 100-150 cm W/SMALLER 1-2 cm DIA.	100		DRILL TIME ≈ 20 MIN. 1 PC. 2.2'
245.0'	7					
	7.2		7.2'-4.7' - GOOD CONCRETE. AIR ENTRP. 3 mm-1 cm DIA., AGG. GOOD DIST., ENTRAIN. AIR 1 mm DIA.	100	2	DRILL TIME ≈ 20 MIN. 1 PC. 2.5'
	10		SEE SHEET 2			

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PROJECT: EISENHOWER '90 HOLE NO. S-19

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 251.5 USGLS | Hole No. S-19

PROJECT: EISENHOWER LOCK | INSTALLATION: | SHEET 2 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			9.7'-14.4' GOOD CONCRETE. ENTRP. AIR 2 mm-2 cm DIA., ENTRAIN AIR ≈ 1 mm THROUGHOUT. ABUNDANT LG. ENTRP. AIR AT 12.7'-14.0', AGG. HAS GOOD DIST.			3 PC. 4.70' 3.60' 1.80' DRILL TIME ~ DRILL ACTION: SMOOTH
	11					
235.0'						
	12				3	
	13					
	14	MB	MB AT BOULDER		14.4	
			14.4'-18.0' GOOD CONCRETE. ABUNDANT AIR ENTRP. 2 mm DIA.-3 cm LONG, AVE. 5 mm DIA.	100		
	15					
	16					
230.0'					4	
	17					
		MB	MB AT BOULDER			
	18		18.0'-19.8' GOOD CONCRETE. SMALLER AGG. AVE. 3-4 cm LONG, ENTRAIN AIR ≈ 1 mm DIA., ENTRP AIR LESS ABUNDANT ≈ 2 mm DIA.			
		MB			18.9	
	19					
			19.8'-21.1' GOOD CONCRETE. ABUNDANT ENTRP. AIR 1-5 mm DIA., AVE. AGG. 2 cm DIA. OCC. LG. AGG. 10 cm			5 PC. 1.30' 5.70' 1.40' 0.80' BROKEN 0.60' 0.20'
	20				5	
	21	MB	MB AT BOULDER			
225.0'						
	22		21.1'-26.8EXITE SHEET 3)			

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DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS	Hole No. S-19
PROJECT EISENHOWER LOCK	INSTALLATION	SHEET 3 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB	21.1'-26.8' GOOD CONC.ETE. ENTRP. AIR 2 mm-1.5 cm DIA. MORE ABUNDANT AT 24.5'-26.8', AGG. HAS GOOD DISTR.		22.8	
	23					
	24					
					6	
	25					
				100		
	26					
220.0'		MB	MB AT BOULDER		26.8	
	27		26.8'-28.8' GOOD CONCRETE. ENTRP. AIR 2 mm-1 cm DIA., AGG. V. SMALL +LG.			
	28	MB				
		MB				
	29		29.0'-29.6' BROKEN CONCRETE. BEATEN OUT OF CORE BARREL.		7	
		MB	29.6'-29.8' GOOD CONCRETE.			
	30		29.8'-33.5' GOOD CONCRETE. LG. HOLE AT 30.8 (2.5 cm SQ.) ABUNDANT ENTRP. AIR 1 cm AVE. DIA. ENTRAIN AIR 1 mm DIA., POSSIBLE COLD AT 30.8'.			7 PC. 2.70, 1.70, 3.20, 1.00, 0.75, 0.35, 0.30
	31	MB			31.1	DRILL TIME ≈ 28 MIN.
215.0'						
	32			100		
		MB	32.5'-34.2' GOOD CONCRETE. ENTRP. AIR 2 mm-1 cm DIA.		8	
	33					
	34					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS	Hole No. S-19
PROJECT EISENHOWER LOCK	INSTALLATION	SHEET 4 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			MB AT BOULDER 34.2'-37.4' GOOD CONCRETE. ABUNDANT ENTRAP. AIR 2 mm-5 mm AVE. 3 mm		8	
	35	MB			35.4	
	36					
210.0'						
	37	MB	MB AT BOULDER 37.4'-39.8' GOOD CONCRETE. ENTRP. AIR LESS ABUNDANT- 5 mm DIA. MAX.	100	9	
	38	MB	MB THROUGH AGG. AGG. GOOD DIST			
	39	MB	MB THROUGH AGG.			
		MB	MB THROUGH AGG. 39.2'-39.8' BEATEN OUT OF END OF CORE BARREL		39.8	
	40		39.8-43.0 GOOD CONCRETE ENTRP. AIR 2 mm-5 mm THRU OUT. AGG. GOOD DIST.			40 MIN. DRILL TIME 39.8'-49.3' 6 PC. 3.20' 3.40' 1.40' 0.90' 0.40' CHIPS 0.20'
	41					
205.0'		MB				
	42				10	
	43	MB	MB AT BOULDER 43.0'-46.4' GOOD CONCRETE. SAME AS ABOVE	100		
	44				44.4	
	45				11	
	46					

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MAR 71

PROJECT	HOLE NO.
EISENHOWER '90	S-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS		Hole No. S-19
PROJECT EISENHOWER LOCK	INSTALLATION		SHEET 5 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
200.0'		MB	BREAK AT BOULDER W/AIR ENTRAIN BUBBLES BROKE			
	47		46.4'-49.4' GOOD CONCRETE. (SAME AS ABOVE)		11	
		MB	BREAK AT BOULDER	100		
	48					
		MB	BREAK AT BOULDER		48.7	
	49	MB	BREAK AT BOULDER			
			49.1'-49.2' BROKEN			
	50		49.4'-54.4' GOOD CONCRETE. ABUNDANT AIR ENTRP. (2 mm-1.5 cm DIA.) AVE. 4 mm AIR ABUNDANT AT 49.4'-50.0'			49.4'-54.4' 2 PC. 3.00' 2.00'
	51				12	
195.0'	52	MB		100	52.4	
	53					
	54					
	55		54.4'-62.9' GOOD CONCRETE. ENTRP. AIR 3 mm-1 cm DIA. AT 54.4'-60.1', MORE ABUNDANT AT 60.1'-62.9' GOOD CONSOLIDATION THRU OUT.		13	54.4'-62.9' 3 PC. 2.90' 2.80' 2.80'
	56					
190.0'	57	MB			57.3	
	58					

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PROJECT EISENHOWER '90 | HOLE NO. S-19

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 USGLS | Hole No. S-19

PROJECT: EISENHOWER LOCK | INSTALLATION: | SHEET 6 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			(SEE SHEET 5)			
	59					
					14	
	60	MB		100		
	61	MB			61.5	
185.0'						
	62					
	63		62.9'-71.1' GOOD CONCRETE. GOOD CONSOLIDATION THRU OUT GOOD AGG. DISTR., ENTRP. AIR 2 mm DIA. 1.2 cm long + irreg.		15	62.9'-72.9' DRILLED 10.0' BUT LEFT 1.8' IN THE HOLE. DRILL TIME 45 MIN.
	64	MB				
	65	MB	66.9'-68.0' ENTRP. AIR MORE ABUNDANT		65.6	
	66					
180.0'		MB		83		
	67				16	
	68					
	69	MB			69.5	
	70					

ENG FORM 1836-A PREVIOUS EDITIONS ARE OBSOLETE.
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PROJECT: EISENHOWER '90 | HOLE NO. S-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS		Hole No. S-19
PROJECT EISENHOWER LOCK		INSTALLATION	SHEET 7 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			(SEE SHEET 6)			
	71	MB	71.1'-78.8' GOOD CONCRETE. GOOD CONSOLIDATION, AIR ENTRP. 2 mm-7 mm DIA. OCC. LG. + IRREG. 2 cm LONG.			
175.0'						
	72				17	
	73					72.9'-79.9' 5 PC. 3.10' 3.40' 1.40' 0.60' 0.30' + CHIPS
	74	MB			74.2	
	75					
	76			126	18	
165.0'						
	77	MB				
	78	LIFT				
	79	MB	78.8-79.9 GOOD CONCRETE. A LITTLE SOFTER, MORE POROUS HOLDS WATER LONGER, CEMENT COLOR IS BROWNISH GRAY W/SMALLER AGG., ENTRP. AIR 2 mm-1 cm DIA.		79.0	
	80		79.0'-79.9' BEATEN OUT OF CORE BARREL		19	79.9'-89.9'
	81		79.9'-82.0' GOOD CONCRETE. SAME AS 79.0'-79.9'. BROWN-GRAY HIGHER CONTENT OF SMALL AGG. 1 mm-2.5 mm IN CEMENT LOOKS WEAKER, POROUS POOR CONSOLIDATION.			
160.0'	82					

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PROJECT EISENHOWER '90 HOLE NO. S-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS	Hole No. S-19
PROJECT EISENHOWER LOCK	INSTALLATION	SHEET 8 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			82.0' GOOD CONCRETE. DARK GRAY SEEMS HARDER GOOD CONSOLIDATION, ENTRP. AIR 2 mm-5 mm DIA.			
	83	MB	MB AT BOULDERS		83.1	
	84	MB	MB AT BOULDERS			
	85	JT	? OR LIFT LINE, COATED W/WHITE MATL.	84	20	
155.0'	86		85.9'-88.6' GOOD CEMENT, COLOR LITTLE BIT LIGHTER THAN ABOVE + MORE POROUS ENTRP. AIR 2 mm-5 mm DIA. CEMENT MATRIX HAS A HIGH CONTENT OF V. SMALL PC. OF AGG.			
	87					LEFT 1.6' IN HOLE
	88	MB			88.3	
		MB	88.3'-98.1' GOOD CONCRETE. SAME AS ABOVE, SOUNDS DRUMMY, ENTRP. AIR V. SMALL 1 mm-4 mm AVE. 2 mm. SOME IRREGULAR BUBS.			
	89					
	90				21	89.9'-98.1' 6 PC. 0.40' 5.30' 2.20' 1.50' 0.35' 0.25'
	91					
150.0'	92	MB		122	92.0	
	93				22	
	94	MB				

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS		Hole No. S-19
PROJECT EISENHOWER LOCK		INSTALLATION	SHEET 9 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			SEE SHEET 8			
	95					
	96	MB			96.2	
145.0'						
	97	MB				
	98	MB			23	
			98.3'-101.6' GOOD CONCRETE. LT. GRAY CEMENT. SAME AS ABOVE. V. HOLEY (ALMOST HONEYCOMB PATTERN). ENTRP. AIR 1 mm-1 cm DIA. ABUNDANT SMALL BUB 1-3 mm.			98.3'-108.2' 6 PC. 3.30' 2.50' 0.90' 1.60' 0.50' 1.10'
	99	MB				
	100	MB			100.4	
	101	MB				
140.0'			101.6'-108.2' GOOD CONCRETE. SAME AS ABOVE. EXCEPT MORE ENTRP. AIR AT 101.6'-103.8'.			
	102			100		
	103				24	
	104					
	105	MB			105.0	
	106					

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PROJECT	HOLE NO.
EISENHOWER '90	S-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS	Hole No. S-19
PROJECT EISENHOWER LOCK	INSTALLATION	SHEET 10 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
135.0'		MB				
	107	MB	VOID AT 106.8' W/8 cm DIA.		25	
	108		108.2'-110.7' GOOD CONCRETE. LT. GRAY CEMENT AS ABOVE. W/CONSIDERABLY LESS ENTRP. AIR 2 mm-1 cm DIA.			108.2'-115.2' 6 PC. 2.50' 0.90' 0.80' 1.40' 0.20' 1.20'
	109					
	110	B			26	
	111		CONCRETE/ROCK CONTACT V. SMOOTH BREAK/JT? NO BOND.	100		
130.0'		MB	111.6' CHIPS			
	112	MB	110.7'-115.2' BLACK GRAY (WET) DOLOMITE INTERBED W/LT. GRAY WEAK CLAY AT 113.8', 114.0'+115'0' 1-3 mm THICK.		112.4	
	113	MB				
	114	MB	DOLOMITE V. HARD, V. DENSE UNIFORM. IN STRUCTURE.		27	
	115	-MB	115.2'-117.0' SAME AS ABOVE			115.2'-120.0' 5 PC.
	116					
125.0'						

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PROJECT EISENHOWER '90 | HOLE NO. S-19

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE 251.5 USGLS | Hole No. S-19

PROJECT EISENHOWER LOCK | INSTALLATION | SHEET 11 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB			117.0	
	117		117.0'-120.0' COLOR CHANGE TO MOTTLED DK. AND LT. DOLOMITE W/NUMEROUS LT. GRAY CLAY THIN LANINAE 1 mm THICK.			
		MB		100		
	118				28	
		MB				
	119	MB				
120.0'						
			LG. CALCITE BLEB.		120.0	
	120		120.0'-125.05' DOLOMITE, BLACK (WET) CALCITE BLEBS THRU OUT, INTERBED W/WEAK BLACK SHALE 4-8 cm THICK. DOLO. V. HARD V. DENSE. STYOLITES COMMON.			120.0'-129.0' 12 PC. 1.60' 0.30' 0.90' 0.10' 0.10' 0.10' MISSING 1.60' 0.30' 0.50' 0.60' 0.90'
	121	MB				
		MB				
	122				29	
		MB				
	123		123.0'-123.1' MISSING.			
	124			98		
115.0'		MB			124.7	
		MB				
	125		125.05'-129.0' DOLOMITE V. HARD V. DENSE, BLACK WHEN WET, LITTLE TO NO SHALE.			
	126				30	
	127	MB				
	128					

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PROJECT EISENHOWER '90 | HOLE NO. S-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5 USGLS		Hole No. S-19
PROJECT EISENHOWER LOCK	INSTALLATION		SHEET 12 OF 12 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
111.0'	129		SEE PREVIOUS SHEET.			BOTTOM OF HOLE.

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PROJECT EISENHOWER '90 | HOLE NO. S-19

Hole No. SS-18

DRILLING LOG	DIVISION	INSTALLATION EISENHOWER LOCK	SHEET 1 OF 4 SHEETS

1. PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT 7¾" OD
2. LOCATION (Coordinates or Station) MONOLITH: SS-18 LOCK STATION 226.00	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL - USGL 251.5'
3. DRILLING AGENCY MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 1500
4. HOLE NO. (As shown on drawing title and file number) SS-18	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED 0 — UNDISTURBED 0
5. NAME OF DRILLER CARL MOON	14. TOTAL NUMBER CORE BOXES 9
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE — STARTED 9 JUNE 90 — COMPLETED 11 JUNE 90
7. THICKNESS OF OVERBURDEN 0	17. ELEVATION TOP OF HOLE 251.5
8. DEPTH DRILLED INTO ROCK 0	18. TOTAL CORE RECOVERY FOR BORING 35.8 %
9. TOTAL DEPTH OF HOLE 35.8'	19. SIGNATURE OF INSPECTOR M.EILEEN GLYN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	0 – 0.7	MB	0.0' - 0.7' GOOD CONCRETE	71		0.0' - 0.7' 1 PC. 0.5'
	0.7 – 3.0	MB	0.7' - 3.0' GOOD CONCRETE LITTLE TO NO ENTRP. AIR / GOOD CONSOLIDATION	115	1	0.7' - 3.0' 1 PC. 2.5'
	3.0 – 4.2	MB	3.0' - 5.3' GOOD CONCRETE. HARD, GOOD CONSOLIDATION, ENTRP. AIR 1mm - 3mm OCC. 7 mm DIA.	100	4.2	3.0' - 5.3' 1 PC. 2.3'
	4.2 – 5.3	MB				
	5.3 – 7.9	MB	BROKE ACROSS AGG. 5.3' - 7.9' GOOD CONCRETE. HARD, GOOD CONSOLIDATION ENTRP. AIR 1 mm - 2 mm DIA.	100	2	5.3' - 7.9' 1 PC. 2.6'
	7.9 – 8.7	MB	7.9' - 9.7' GOOD CONCRETE. (SAME AS ABOVE)	45	8.7	7.9' - 10.4' 1 PC. 1.8'
	8.7 – 9.7	LIFT			3	
	9.7 – 10	?				

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PROJECT EISENHOWER R '90 — HOLE NO. SS-18

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. SS-18
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 2 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			9.7' - 19.6' GOOD CONCRETE GOOD CONSOLIDATION / ENTRP. AIR 1 mm - 3 mm DIA. NOT ABUNDANT. OCC. 1 cm DIA.			10.4' - 19.7' 2 PC. 5.3' 4.6'
	11				3	
	12	MB			12.5	
	13				4	
	14	MB				
	15	LIFT ?	BREAK IS RELATIVELY PLANER.	107	15.0	
	16		STEEL AT. 15.8' 8 cm LONG, 2 WIDE			
	17				5	
	18	MB				
	19				19.7	
	20		19.7' - 25.8' GOOD CONCRETE. GOOD CONSOLIDATION LITTLE ENTRP. AIR			19.7' - 25.8' 1 PC. 6.2'
	21				6	
	22					

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PROJECT EISENHOWER R '90

HOLE NO. SS-18

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. SS-18
PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION: EISENHOWER LOCK	SHEET 3 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	23				6	
	23.6	MB		100		
	24					
	24.5	MB				
	25					
	25.8	MB				
	26		25.8' - 34.7' GOOD CONCRETE. AS ABOVE.		7	25.8' - 35.8' 1.6' 2.9' 4.3' 0.7' 0.4' 0.1' 10.0'
	27	MB			27.4	
	28					BOTTOM OF RUN WAS BEATEN OUT OF CORE BARREL
	29				8	
	30			100		
	30.3		CRACK ? OR LIFT ? NO EASILY IDENTIFIABLE WEATHERING, NO STAINING.			
	31					
	31.9	MB			31.9	
	32					
	33	MB			9	
	34					

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PROJECT: EISENHOWE R '90 | HOLE NO. SS-18

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. SS-18
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			SEE PREVIOUS SHEET			
	34	MB			9	
	35					35.8' BOTTOM
	36					
	37					
	38					
	39					

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PROJECT EISENHOWER R '90

HOLE NO. SS-18

Hole No. SS-19

DRILLING LOG	DIVISION	INSTALLATION EISENHOWER LOCK	SHEET 1 OF 4 SHEETS
1. PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS		10. SIZE AND TYPE OF BIT 7 3/4" DIAMON	
2. LOCATION (Coordinates or Station) MONOLITH: SS-19 LOCK STATION 275.00		11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL - USLS	
3. DRILLING AGENCY MOBILE DISTRICT		12. MANUFACTURER'S DESIGNATION OF DRILL FAILING 1500	
4. HOLE NO. (As shown on drawing title and file number)	SS-19	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED 0 / UNDISTURBED 0
5. NAME OF DRILLER CARL MOON		14. TOTAL NUMBER CORE BOXES 8	
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ______ DEG. FROM VERT.		15. ELEVATION GROUND WATER	
7. THICKNESS OF OVERBURDEN 0		16. DATE HOLE	STARTED 11 JUNE 90 / COMPLETED 12 JUNE 90
8. DEPTH DRILLED INTO ROCK 0		17. ELEVATION TOP OF HOLE 251.5	
9. TOTAL DEPTH OF HOLE 35.0'		18. TOTAL CORE RECOVERY FOR BORING 35.0 %	
		19. SIGNATURE OF INSPECTOR M.EILEEN GLYN	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	0–2.5	MB	0.0' - 2.5' GOOD CONCRETE. SOME ENTRP. AIR 1 mm - 3 mm DIA. GOOD CONSOLIDATION	100		0.0' - 2.5' 2 PC. 2.2' 0.2' BOTTOM BEATEN OUT OF CORE BARREL
	2.4–4.4	MB MB	2.4' - 4.4' GOOD CONCRETE. SAME AS ABOVE	80	1	2.4' - 5.0' 1 PC. 2.0'
	4.4–6.9		4.4' - 6.9' GOOD CONCRETE. SAME AS ABOVE		4.9	
		MB		120		5.0' - 7.5' 2 PC. 1.8' 0.5'
	6.9–9.4	MB	6.9' - 9.4' GOOD CONCRETE. SAME AS ABOVE	120	2	7.5' - 9.4'
		MB			9.4	
	10					9.4' - 19.1'

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PROJECT EISENHOWER '90 | HOLE NO. SS-19

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE	Hole No.
251.5	SS-19

PROJECT	INSTALLATION	SHEET
EVALUATION OF EISENHOWER AND SNELL LOCKS	EISENHOWER LOCK	SHEET 2 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			9.4' - 19.1' GOOD CONCRETE, ENTRP. AIR 1 mm - 3mm DIA. GOOD CONSOL. + GOOD BONDING W/ AGG.			3 PC. 4.6' 3.7' 1.4' = 9.7'
	11	MB				
	12				3	
	13					
	14	MB	BREAK AT LG. BOULDER.			
	15		ELONGATED PC. STEEL. 8 cm LONG.	100		
	16	MB			4	
	17					
		MB	BREAK AT LG. BOULDER			
	18	MB				
	19	MB	BREAK AT LG. BOULDER			
			19.1' - 26.1' GOOD CONCRETE. ENTRP. AIR 1 mm - 3 mm DIA. OCC. IRREG. BUBS 1 cc LONG. AIR MORE ABUNDANT THAN ABOVE. GOOD CONSOL.			19.1' - 26.0'
	20				5	
	21			100		
	22					

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PROJECT	HOLE NO.
EISENHOWER '90	SS-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. SS-19
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 3 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	23	MB			23.1	
	24					
	25				6	
	26	O	26.1' - 35.0' GOOD CONCRETE. ENTRP. AIR 1 mm - 5 mm DIA. NOT ABUNDANT. GOOD CONSOLIDATION, V. HARD			26.1' - 35.0' 2 PC. 7.0' 2.0'
	27					NO EVIDENCE OF A CRACK 26.1' - 35.0'
	28	MB			28.3	
	29					
	30	MB	BROKE WHEN BROUGHT OUT OF BARREL. PROBABLY A 1FT JT. IF ANYWAY.		7	
	31					
	32				32.5	
	33	MB	BREAK AT LG. BOULDER.		8	
	34					

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PROJECT EISENHOWER '90 | HOLE NO. SS-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5		Hole No. SS-19
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK		SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
					8	
	35	MB				35.0' BOTTOM

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PROJECT EISENHOWER '90

HOLE NO. SS-19

Hole No. SN-N56

DRILLING LOG	DIVISION	INSTALLATION SNELL LOCK	SHEET 1 of 7 SHEETS

1. PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station): MONOLITH: SN-N56 LOCK STATION 462.00	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL - USLS 205'
3. DRILLING AGENCY: MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 1500
4. HOLE NO. (As shown on drawing title and file number): SN-N56	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 19 JUN 90; UNDISTURBED
5. NAME OF DRILLER: CARL MOON	14. TOTAL NUMBER CORE BOXES: 18
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE: STARTED 19 JUNE 90; COMPLETED 21 JUNE 90
7. THICKNESS OF OVERBURDEN: 0	17. ELEVATION TOP OF HOLE: 205'
8. DEPTH DRILLED INTO ROCK: 0	18. TOTAL CORE RECOVERY FOR BORING: 73.9 %
9. TOTAL DEPTH OF HOLE: 73.9'	19. SIGNATURE OF INSPECTOR: M.EILEEN GLYNN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	0–2.6		0.0' - 2.6' GOOD CONCRETE. ENTRP. AIR ABUNDANT 1 mm - 2 mm DIA. OCC. 5 mm - 1 cm DIA. IRREG. PITTED TEXTURE THRU OUT.		1	PULL 1 0.0' - 2.6' 1 PC. 2.6'
	2.6	MB	BREAK AT BOULDER			
	2.6–4.7		2.6' - 4.7' GOOD CONCRETE. ENTRP. AIR 1 mm - 3 mm ABUNDANT. OCC. 5 mm - 2 cm IRREG. LONG BUBS.			PULL 2 2.6' - 5.0' 2 PC. 1.7' 0.4' 2.2' LOSS 0.3'
	4.3	MB			4.3	
	4.7	MB				
	4.7–9.6		4.7' - 9.6' GOOD CONCRETE, ABUNDANT. ENTRP. AIR 1 mm - 2 mm OCC. 4 mm DIA., HARD, SOLID.		2	PULL 3 5.0'- 7.1' 1 PC. 2.4' REC. 2.4' LOSS 0.0' GAIN 0.0'
	7.1	MB	PLANAR BREAK.			PULL 4 7.1'- 9.6' 2 PC. 1.5' 0.9' REC. 2.4' LOSS 0.1' GAIN 0.0'
	8.7	MB	— BREAK AT LG. BOULDER		8.7	
	9.6	MB				
	10					

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 251.5	Hole No. SS-19
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION EISENHOWER LOCK	SHEET 4 OF 4 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
					8	
	35	MB				35.0' BOTTOM

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PROJECT EISENHOWER '90 HOLE NO. SS-19

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-N56
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK	SHEET 2 OF 7 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			9.6' - 19.6' GOOD CONCRETE. ABUNDANT. ENTRP. AIR 1 mm - 2 mm DIA. OCC. 5 mm DIA. RELATIVELY SMOOTH TEXTURE GOOD CONSOL. HARD.			9.9' - 19.6' 2 PC. 2.6' 5.1' 2.3' REC. 10.0' LOSS 0.0' GAIN 0.0'
	11				3	
	12	? LIFT	— PLANAR BREAK			
		MB			12.9	
	13					9.6' 2.6' 12.2'
	14					
	15				4	
	16					
	17	? LIFT	— PLANAR BREAK		17.3	
	18	MB				
	19	MB				
		MB			5	
			19.6' - 27.3' GOOD CONCRETE. HARD, GOOD CONSOL. ABUNDANT SMALL ENTRP. AIR. 1 - 2 mm DIA. OCC. 3 mm DIA. - 1 cm IRREG.			19.6' - 29.3' 4 PC. 2.6' 3.3' 1.8' REC. 7.7' LOSS 2.0' GAIN 0.0'
	20					
	21	MB				
		MB			21.7	
	22					

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PROJECT EISENHOWER '90 | HOLE NO. SN-N56

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-N56
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK	SHEET 3 OF 7 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB				
	23					
					6	
	24					
	25	? LIFT				
		MB	— PLANAR BREAK		25.5	
	26					
	27	MB				
			27.3' - 37.0' GOOD CONCRETE, HARD. GOOD CONSOL, REL. SMOOTH TXT. ENTRP. AIR. 1 mm DIA. OCC. 2 - 5 mm DIA.		7	37.0' 27.3' 9.7'
	28					
	29	MB			29.6	
	30					29.3' - 37.0' 1 PC. 5.1' 4.6' REC. 3.7' GAIN 2.0' LOSS 0.0'
	31					
					8	
	32	MB				
	33					
		MB			33.7	
	34					

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PROJECT EISENHOWER '90 | HOLE NO. SN-N56

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE	Hole No.
205	SN-N56

PROJECT	INSTALLATION	SHEET
EVALUATION OF EISENHOWER AND SNELL LOCKS	SNELL LOCK	4 OF 7 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	35	MB			9	
	36					
	37	MB	37.0' - 44.7' GOOD CONCRETE, GOOD CONSOL, ENTRP. AIR 1 - 3 mm DIA., SOME IRREG, 4 mm - 2.5 cm LONG, REL. SMOOTH TXT.		37.7	37.0' - 45.8' 2 PC. 3.3' 4.4' 0.1' CHIPS 0.02' REC. 8.0' LOSS 0.8' GAIN 0.0'
	38	MB				
	39				10	
	40	MB		8.0 / 8.8		
	41	MB			41.7	
	42					
	43	MB				
	44				11	
	45	MB	LARGE VOIDS AT 44.7' — BREAK AT VOIDS — LIFT ?			
	46		45.0' - 54.5' (SEE NEXT SHEET)			

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PROJECT	HOLE NO.
EISENHOWER '90	SN-N56

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-N56
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK	SHEET 5 OF 7 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	47		45.0' - 54.5' GOOD CONCRETE, GOOD CONSOL, HARD. AIR ENTRP. ABUNDANT 1 mm DIA. THRU OUT 2 mm - 1 cm FREQ., REL. SMOOTH TXT.			45.8' - 54.5' 2 PC. 6.4' 3.1' REC. 9.5' LOSS 0.0' GAIN 0.8'
	48				12	
	49					
	50	50.			50.0	
	51					
	52				13	
	53	MB				
	54				54.5	
	55		54.5' - 64.1' GOOD CONCRETE, HARD, GOOD CONSOL. ENTRP. AIR 1 - 3 mm DIA. PITTED TXT. THRU OUT. SOME 5 mm - 1 cm IRREG.			54.5' - 64.1' 4 PC. 1.2' 3.5' 3.3' 1.4' REC. 9.4' LOSS 0.2' GAIN 0.0'
	56				14	
	57	MB				
	58					

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PROJECT EISENHOWER '90 | HOLE NO. SN-N56

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205.0'		Hole No. SN-N56
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK		SHEET 6 OF 7 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB			58.7	
	59	MB				
	60					
					15	
	61					
	62	MB			62.6	BOTTOM 2.0' BEATEN OUT OF BARREL.
	63	MB				
						64.1'
	64		63.9' - 73.9' GOOD CONCRETE, ABUNDANT ENTRP. AIR AIR 1 mm - 3 mm DIA. FREQ. 4 mm - 1 cm IRREG. BUB. PITTED TEXTURE, SOME MINOR DEGRADATION AROUND AGG.		16	64.1' - 73.9' 3 PC. 3.2' 5.9' 0.9' 10.0'
	65					
	66					
	67	MB			67.1	
	68					
	69	MB				
	70					

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PROJECT EISENHOWER '90 | HOLE NO. SN-N56

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205.0'	Hole No. SN-N56
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK	SHEET 7 OF 7 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	71	MB			71.6	
	72				18	
	73	MB				
	74					37.9' BOTTOM

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PROJECT EISENHOWER '90 | HOLE NO. SN-N56

Hole No. SN-S15

DRILLING LOG	DIVISION	INSTALLATION SNELL LOCK	SHEET 1 OF 9 SHEETS

1. PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS	10. SIZE AND TYPE OF BIT
2. LOCATION (Coordinates or Station): MONOLITH: SN-S15 LOCK STATION 319.00	11. DATUM FOR ELEVATION SHOWN (TBM or MSL): MSL - USLS 205'
3. DRILLING AGENCY: MOBILE DISTRICT	12. MANUFACTURER'S DESIGNATION OF DRILL: FAILING 1500
4. HOLE NO. (As shown on drawing title and file number): SN-S15	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED: 0 — UNDISTURBED: 0
5. NAME OF DRILLER: CARL MOON	14. TOTAL NUMBER CORE BOXES: 24
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
7. THICKNESS OF OVERBURDEN: 0	16. DATE HOLE — STARTED: 13 JUNE 90 — COMPLETED: 18 JUNE 90
8. DEPTH DRILLED INTO ROCK: 7.2	17. ELEVATION TOP OF HOLE: 205
9. TOTAL DEPTH OF HOLE: 106.2'	18. TOTAL CORE RECOVERY FOR BORING: 104 %
	19. SIGNATURE OF INSPECTOR: M.EILEEN GLYNN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	0.0–2.5		0.0' - 2.5' GOOD CONCRETE. GOOD CONSOLIDATION ENTRP. AIR 1 mm - 2 mm DIA. OCC. 1 cm DIA.	100	1	PULL 1 0.0 - 2.5' REC. 2.5' LOSS 0 1 PC. 2.5' 13 JUNE 1030 HRS
	2.5–5.0	MB	2.5' - 5.0' GOOD CONCRETE. GOOD CONSOLIDATION ENTRP. AIR 1 mm - 3 mm DIA.		4.5	PULL 2 14 JUNE 90 1600 HRS 2 PC. 2.0' 0.5' REC. 2.5' LOSS 0.0
	5.0–6.4	MB	— LIFT ? 5.0' - 6.4' GOOD CONCRETE.			5.0' - 7.5' PULL 3 2 PC. 1.4' 1.1'
	6.4–7.5		6.4' - 7.5' RUBBLE BROKE OUT PC. OF REBAR.		2	1.1 LENGTH OF REBAR. THREW 1.1 AWAY.
	7.5–9.9		7.5' - 9.9' GOOD CONCRETE, ABUNDANT. AIR ENTRP. 1 mm - 3 mm DIA. GOOD CONSOLIDATION, HARD.	100	9.9	PULL 4 7.5' - 9.9'

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

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PROJECT: EISENHOWER '90 — HOLE NO. SN-S15

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-S15
PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION: SNELL LOCK	SHEET 2 OF 9 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			9.9' - 19.4' GOOD CONCRETE, ABUNDANT. ENTRP. AIR 1 mm - 3 mm DIA. OCC. IRREG. BUB 1-2 cm LONG. COLOR UNIFORM THRU OUT. (GREY)			9.9' - 19.6' 2 PC. 7.1' 2.4' REC. 9.5' LOSS 0.1'
	11					
	12	MB			3	PULL 5
	13	MB				
	14	MB			14.8	
	15					
	16					
	17	MB	BREAK AT LG. BOULDER WHITE RXN. MATL. ON BREAK.		4	
	18	MB	BREAK AT LG. BOULDER WHITE RXN. MATL. ON BREAK.			
		MB				
	19				19.3	
			19.3' - 29.2' GOOD CONCRETE, HARD. GOOD CONSOLIDATION, ABUNDANT ENTRP. AIR 1 mm - 3 mm DIA. (LESS ABUNDANT 19.3' - 22.4')			19.6' - 29.2' 3 PC. 5.5' 2.6' 1.8' 9.9'
	20					
					5	PULL 6 REC. 9.9' LOSS 0.0' GAIN 0.3'
	21					
	22	MB				

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PROJECT: EISENHOWER '90 — HOLE NO. SN-S15

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205		Hole No. SN-S15
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK		SHEET 3 OF 9 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	23				5	
		MB			23.9	
	24					
		MB				
	25					
	26				6	
	27	MB				
	28				28.0	
	29		29.2' - 38.8' GOOD CONCRETE, MINOR DEGRADATION AROUND AGG. SHALLOW PITTED TEXTURE THROUGHOUT, ENTRP. AIR 1 mm - 3 mm ABUNDANT, OCC. LCM IRREG. BUB.			29.2' - 38.8' 2 PC. 3.2' 6.4' 9.6' REC. 9.6' PULL 7
	30				7	
	31			100		
	32	MB	— BREAK IS PLANAR.		32.4	
	33					
	34					

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 205 | Hole No. SN-S15

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: SNELL LOCK | SHEET 4 OF 9 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
		MB				
	35				8	
	36	MB				
		MB			36.9	
	37	MB				
	38					HOLE
		MB	BREAK AT LG. BOULDER			
	39		38.8' - 42.8' GOOD CONCRETE, MINOR DEGRADATION AROUND AGG. ENTRP. AIR 1 mm - 3 mm OCC. 5 mm DIA. RELATIVELY SMOOTH TEXTURE.		9	38.8' - 48.3' REC. 1 PC. 8.8' LOSS 0.7' GAIN 0.0'
	40					PULL 8
	41	MB			41.3	
	42					
	43		42.8' - 47.6' GOOD CONCRETE, A LITTLE MORE DEGRAD. AROUND AGG. THAN ABOVE, PITTED TEXTURE. ENTRP. AIR 1 mm - 3 mm		10	
	44	MB	LIFT ? 44.8' - 47.6'			
	45					
	46	MB			45.9	

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PROJECT: EISENHOWER '90 | HOLE NO. SN-S15

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 205

Hole No. SN-S15

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS

INSTALLATION: SNELL LOCK

SHEET 5 OF 9 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	47		LG. VOIDS AT 47.5' ≈ 2 cm DIA. BREAK AT BOULDER			
	48		47.6' - 49.6' GOOD CONCRETE, MINOR DEGRADATION AROUND AGG. ENTRP. AIR 1 mm - 3 mm DIA.		11	48.3' - 56.6' PULL 9
	49					
	50		49.6' - 51.1' POOR CONCRETE, LG. VOIDS UPTO 10 cm LONG. CORE NOT INTACT 50.4' - 50.7'		50.4	REC. 8.0' GAIN 0.7' LOSS 1.0'
	51		51.1' - 55.6' GOOD CONCRETE, RELATIVELY SMOOTH TEXTURE. ENTRP. AIR. 1 mm - 3 mm DIA.		51.1	2 PC. 2.9' 5.1'
	52	MB				
	53				12	PROBABLY ONLY RECOVERED 7.5' ?
	54	MB				
	55		55.0' - 55.6' IRREG ENTRP. AIR AROUND AGG.		55.2	
	56		55.6' - 57.8' GOOD CONCRETE, MINOR DEGRADATION AROUND AGG. ENTRP. AIR 1 mm - 3 mm DIA.		13	56.6' - 64.6' 2 PC. 2.2' 5.2' REC. 7.4' GAIN LOSS PULL 10
	57					
	58	MB	LG. VOIDS AT 57.8'			

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PROJECT: EISENHOWER '90

HOLE NO. SN-S15

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-S15
PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION: SNELL LOCK	SHEET 6 OF 9 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
	59		57.8' - 63.1' GOOD CONCRETE, MINOR DEGRADATION AROUND AGG. ENTRP. AIR 1 mm - 3 mm DIA.		13	
	60				59.9	
	61	MB	— LIFT ? 60.8' - 63.1' SLIGHTLY SMOOTHER TEXTURE			
	62	MB, MB			14	
	63	MB	63.1' - 72.1' GOOD CONCRETE, ENTRP. AIR 1 mm - 2 mm OCC. 3 mm - 1 cm DIA. GOOD CONSOLIDATION			
	64	MB			64.4	
	65		63.1' - 64.1' AND 65.8' - 67.8', HIGH CONTENT OF SMALL AGG. 3 mm TO 1 cm LONG.			64.6' - 72.1' PULL 11 3 PC. 4.7' 4.3' 0.8' REC. 9.8' LOSS 0.0' GAIN 2.3' MEASURED DEPTH OF HOLE 72.1' DOES NOT CORRELLATE W/ AMT. OF CORE. 72.9', PROBABLY MEASURED PULLS 8' - 9' LONG.
	66				15	
	67					
	68				68.1	
	69				16	
	70					

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PROJECT: EISENHOWER '90 HOLE NO. SN-S15

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: 205 | Hole No. SN-S15

PROJECT: EVALUATION OF EISENHOWER AND SNELL LOCKS | INSTALLATION: SNELL LOCK | SHEET 7 OF 9 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
			LG. BOULDER 8 IN. LONG AT 70.5'			
	71	MB			16	
	72				72.1	
			72.1' - 81.0' GOOD CONCRETE, HARD. GOOD CONSOL. ENTRP. AIR 1 mm - 4 mm OCC. 1 cm - 1.5 cm DIA. TEXTURE REL. SMOOTH.			72.1' - 81.0' PULL 12 MONDAY MORNING 3 PC. 0.9' 5.6' 2.4' REC. 8.9' LOSS 0.0' GAIN 0.0'
	73	MB	V. LITTLE. ENTRP. AIR 72.1' - 73.1'			
	74				17	BOTTOM WAS BEATEN OUT OF CORE BARREL EXT.
	75					
	76	MB			76.3	
	77					
	78	MB	— BREAK AT LG. BOULDER		18	
	79	MB				
	80	MB			80.5	
	81		81.0' - 90.7' (SEE NEXT SHEET)		19	PULL 13 81.0' - 90.8' REC. 9.7' LOSS 0.1' GAIN 0.0'
	82					

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DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-S15
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK	SHEET 8 OF 9 SHEETS

ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant)
		MB	81.0' - 90.7' GOOD CONCRETE, HARD. GOOD CONSOL. SMOOTH TEXTURE, ENTRP. AIR 1 mm - 2 mm DIA. OCC. 1 cm DIA. IRREG. BUB.			REC. 3 PC. 4.6' 1.4' 1.0' 2.0' } 3.7' 0.7' 9.7'
	83				19	
	84	MB			84.7	
			STEEL AT 84.8' 2.5 cm DIA.			
	85	MB	— POSSIBLE LIFT ? RELATIVELY PLANAR BREAK.			
	86				20	
	87	MB	— BREAK AT LG. BOULDER			
	88	MB				
		MB			88.5	
	89					
	90	MB				
		MB	— BREAK AT LG. BOULDER		21	
	91		90.7' - 98.6' GOOD CONCRETE, SMOOTH TEXTURE ENTRP. NOT ABUNDANT 1 mm - 3 mm DIA. OCC. 1 cm DIA. LARGE VOIDS AT BREAK ABOUT 0.2' THICK AT 93.7'			90.8' - 100.2' PULL 14 3.0' 3.8' 1.6' 0.7' 0.2' CHIPS 0.2' CHIPS
	92	MB MB			92.4	
	93				22	REC. LOSS 0.0' GAIN 0.3'
	94					

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PROJECT EISENHOWER '90 | HOLE NO. SN-S15

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE 205	Hole No. SN-S15
PROJECT EVALUATION OF EISENHOWER AND SNELL LOCKS	INSTALLATION SNELL LOCK	SHEET 9 OF 9 SHEETS

ELEVATION	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	% CORE RECOVERY	BOX OR SAMPLE NO.	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant)
	95					
	96	MB			96.2	
	97	MB	— BREAK AT LG. BOULDER			
	98				23	
	99	MB	ROCK/CONCRETE INTERFACE AT 99.1' NO BOND. 99.1' - 100.2' V. HARD. GRAY DOLOMITE LOOKS LAMINATED			
		MB				BOTTOM OF RUN WAS BEATEN OUT OF CORE BARREL. (ORIG. ONE PC.)
	100	MB			100.2	
		MB				
		MB	100.2' - 103.3' GOOD, V. HARD DOLOMITE CRYSTALLINE, LAMINATED			100.2' - 106.2' PULL 15 REC. 4.1'
	101					0.2' 2.6' 0.3' 0.3' 0.3' 0.4'
	102				24	4.1' LOSS 2.0'
		MB				
	103	MB				
		MB	103.3' - 104.3' SHALEY DOLOMITE WORM-LIKE VEINS OF SOFTER MATL.			
	104				104.3	
	105					
	106					

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PROJECT EISENHOWER '90

HOLE NO. SN-S15

APPENDIX C: DESCRITPTIONS OF *SOILSTRUCT*

SOIL-STRUCT: Finite Element Computer Program for Soil-Structure Interaction Analysis

SOILSTRUCT, is a general purpose finite element program for two-dimensional, plane strain analysis of soil-structure interaction and soil-inclusion interaction problems. It calculates displacements and stresses due to incremental construction and/or load application and is capable of modeling nonlinear stress-strain material behavior. The simulation of incremental construction may include embankment construction or backfilling, the placement of layer(s) of a reinforcement material during backfilling or embankment construction, dewatering, excavation, installation of a strut or tie-back anchor excavation support system, removal of the same system and the placement of concrete or other construction materials. The incremental loading simulation may consist of the application of concentrated loads, boundary pressures or loads due to temperature changes in non-soil materials.

The initial version of SOILSTRUCT was developed by Professors G. W. Clough and J. M. Duncan for use in the analysis of Port Allen and Old River U-frame locks, Clough and Duncan (1969). This version of the program reflects modifications made in conjunction with a number of projects at WES to expand the capabilities of the finite elements constitutive models, load vector formulation algorithms, the size of problem which may be analyzed, and the transfer of input, output, restart and plot data files by means of disc storage.

FINITE ELEMENTS EMPLOYED

Three types of finite elements are used to represent the behavior of different materials, (1) a two-dimensional continum element, (2) an interface element, and (3) a one-dimensional bar element.

A two-dimensional, subparametric, quadrilateral element (QM5) is used to represent the soil and most structural materials. Structural supports, such as the struts or tieback components of an excavation support system, are typically modeled as a spring support using bar elements. However, two-dimensional elements have been used to model these supports. The geometry of this element, developed by Doherty, Wilson, and Taylor (1969), is defined by four external nodes, while the displacement functions include an internal fifth node. To improve flexural response, a constant shear strain, calculated at the location of the internal fifth node, is imposed throughout the element. The QM5 element can be allowed to degrade to a triangular element by letting two adjacent nodes of the quadrilateral coincide.

The Goodman, Taylor, and Breeke (1969) interface element is used to allow for relative movement between different materials, such as between a soil backfill and a support wall. This element is defined by four nodes, with each of the two pairs of nodes having the same coordinates; thus, this type of element has no thickness.

One-dimensional, two node bar or spring elements are used to model the behavior of a variety of structural systems. This includes the modeling of structural supports such as braces or tiebacks or the modeling of reinforcement placed within a soil backfill.

MATERIAL STRESS-STRAIN BEHAVIOR

Several modes of stress-strain behavior are utilized to represent the response of soil, construction materials and the interface region between different materials.

The constitutive relationship used for all two-dimensional elements is Hooke's law. SOILSTRUCT uses an incremental, equivalent linear method of analysis to model nonlinear material behavior. In this type of analysis, the incremental changes in stresses are related to the incremental strains through a linear relationship. This relationship is defined for each structural element by two engineering constants, the Youngs moduli and the Poisson's ratio. For the soil elements either the Youngs moduli and Poisson's ratio, or the Youngs moduli and bulk moduli may be specified.

Non-Linear Stress-Strain Response of Soil

A plane strain, isotropic drained or undrained stress-strain soil model is incorporated within SOILSTRUCT. The program uses a nonlinear, stress-dependent hyperbolic curve to represent the relationship between stress and strains developing during primary loading of the soil (Figure 1a) and a linear stress-strain response during unloading or reloading of the soil (Figure 1b). The unload-reload stress-strain response is applicable when the current stress state is less than that which has been applied previously; otherwise, the primary loading stress-strain is appropriate. Laboratory testing and interpretation procedures for determining the parameters used to define the soil model are described in Duncan, Byrne, Wong, and Mabry (1978). A brief review of the hyperbolic model is given below.

The nonlinear soil response to loading is modeled by performing a series of analyses in which each load is applied incrementally, with the total change in stress computed at the center of each soil element being equal to the sum of the incremental changes in stress over all the load steps. In general, the greater the curvature of the stress-strain relationship or the larger the magnitude of the applied load, the greater

the number of load steps required to accurately model the nonlinear soil response. This may be achieved in two ways using SOILSTRUCT; either the total load is applied using a greater number of incremental loadings, or during the course of each load case analysis the load vector may be applied in series of increments using the substep option.

Application of each loading in the finite element analysis results in a change in stress within each of the soil elements. In addition to the change in stress, there is a corresponding change in stiffness. Since each incremental analysis is performed assuming equivalent linear element response, SOILSTRUCT updates the value of the elastic moduli assigned to each soil element so as to reflect the magnitude of the current stress state within the element. To account for the change in stiffness that occurs during the application of a load increment, each incremental load calculation may be repeated using the iteration option. When the iteration option is invoked, the load vector is reapplied with a revised value for the element stiffness. The value assigned for the stiffness of the soil element reflects the average of the stress state developed at the end of the previous load case, or substep, and that which develops during the current iteration. However, when only one iteration is specified, the modulus values are calculated using the stresses developed at the end of the previous the load increment. Upon completion of the last iteration for each load case or substep, the arrays tabulating the values of the total nodal point displacements and total element stresses are updated with the computed incremental values.

Primary Loading - Youngs Moduli

Prior to each analysis a tangent Youngs moduli, E_t, is assigned to each soil element. The stress-dependent value of E_t is computed using the relationship

$$E_t = E_i(1 - R_f SL)^2$$

where

E_i = the initial Youngs moduli
R_f = the failure ratio
SL = the stress level

The initial Youngs moduli, E_i, is equal to

$$E_i = KP_a\left(\frac{\sigma_3}{P_a}\right)^n$$

K = the modulus number
n = the modulus exponent
P_a = Atmospheric Pressure
σ_3 = minor principal stress

The proportion of mobilized shear strength for each soil element is reflected in the value of the stress level, SL. SL is equal to the current deviator stress, $(\sigma_1 - \sigma_3)$, divided by the deviator stress at failure, $(\sigma_1 - \sigma_3)_f$, denoted by the subscript f.

$$SL = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f}$$

where

σ_1 = major principal stress

The value of SL ranges from a value equal to zero, to a value equal to unity. SL equal to zero indicates an isotropic stress state, while SL equal to unity corresponds to the complete mobilization of shear resistance within the soil element.

The version of SOILSTRUCT used in this study allows for two procedures for defining the deviator stress at failure; the original Duncan formulation, Duncan and Chang (1970), and a procedure developed by Peters(1988). In the original Duncan formulation as shown in Figure 2a, the values of the minor principal stress at failure is set equal to the current minor principal stress. The deviator stress at failure is given by

$$(\sigma_1 - \sigma_3) = \frac{2c\cos\phi + 2\sigma_3\sin\phi}{1 - \sin\phi}$$

where

c = cohesion intercept
ϕ = angle of internal friction

In the Peters formulation shown in Figure 2b, the value for the deviator stress at failure is defined based upon the assumption that the average value of the major and minor principal stresses at failure is equal to the average value of the current major and minor principal stresses.

$$(\sigma_1 - \sigma_3) = 2c\cos\phi + (\sigma_1 + \sigma_3)\sin\phi$$

The difference between these two procedures may be expressed in terms of the resulting vector curves as shown in Figure 2. Vector curves are loci of points describing the state of stress on planes on which failure eventually occurs. The resulting vector curves in the Duncan formulation are to the right; reflecting the assumption that failure is a result of an increase in major principal stress. In contrast, the vector curves to the left in the Peters formulation are attributed to a coincident increase in major principal stress and decrease in the minor principal stress. In general, the Duncan formulation results in larger values of $(\sigma_1 - \sigma_3)_f$ and therefore smaller values of SL than the Peters formulation. The Peters formulation was developed for undrained loading where $\sigma_1' + \sigma_3'$ is the constant consolidation stress determined for the initial stress state.

The failure ratio, R_f, relates the ultimate deviator stress, $(\sigma_1 - \sigma_3)_{ult}$, to the deviator stress at failure , $(\sigma_1 - \sigma_3)_f$.

$$(\sigma_1 - \sigma_3)_f = R_f(\sigma_1 - \sigma_3)_{ult}$$

The ultimate deviator stress is the asymptote to the stress-strain hyperbola, as shown in Figure 1a. The value of R_f is always less than unity and varies from 0.5 to 0.9 for most soils.

Unload-Reload Stress-Strain Behavior

During unloading or reloading when the current deviator stress is less than that which has been applied during previous loadings, a stress dependent, linear response is assumed, as shown in Figure 1b. In this case, the value of E_{ur} is computed using

$$E_{ur} = K_{ur} P_a \left(\frac{\sigma_3}{P_a}\right)^n$$

where

K_{ur} = the unload-reload modulus number

Poisson's Ratio

The second elastic parameter used to define the material behavior of soil is either the Poisson's ratio, ν, or the bulk moduli, B. This version of SOILSTRUCT allows either parameter to be used. When using Poisson's ratio two values are specified; a constant value which is applicable for all states of stress prior to failure, SL < 1, and the value of Poisson's ratio applicable when the shear strength of the soil is fully mobilized, SL = 1.

The hyperbolic model is designed so that as the soil approaches failure the 0.5. The variation in ν is accomplished by computing the shear modulus with E_t and ν_i and bulk modulus with E_i and ν_i. This variation in ν amounts to

$$\nu = \frac{1}{2}\left(\frac{1-b}{1+\frac{1}{2}b}\right)$$

where

$$b = \left(\frac{E_t}{E_i}\right)\left(\frac{1-2\nu_i}{1+\nu_i}\right)$$

Therefore, $\nu = \nu_i$ when $E_t = E_i$ but increases toward 0.5 as E_t becomes small, near failure.

Interface Response

Interface elements are used to allow for relative movement between different material regions, such as between a soil backfill and a support wall. These elements

are defined by four nodes, each node having two degrees of freedom and each of the two pairs of nodes share the same coordinates. The interface element, therefore, is of finite length but zero thickness.

The properties of interface elements are defined by an interface normal stiffness, k_n, and an interface shear stiffness, k_s. These values of stiffness relate the average relative displacements normal to the interface element, Δ_n, and average relative shear displacements, Δ_s, to the corresponding normal stress, σ_n, and shear stress, τ, by the equations

$$\sigma_n = k_n \Delta_n$$

and

$$\tau = k_s \Delta_s$$

The units of k_n and k_s are force per cubic length.

The value of k_n is set equal to 1 X 10^8 within the program. This value for k_n ensures that the normal relative displacement of the interface element is insignificant when English units (ft, lbs) or SI units (m, KN) are used. If other units are used, the value of the normal stiffness may need to be changed to a higher value.

Two types of interface shear response are modeled, a bilinear shear stress-displacement relationship shown in Figure 3a, and a hyperbolic shear stress-displacement relationship shown in Figure 3b. In the bilinear model, the value assigned to k_s is a constant so long as the average shear stress, τ, along the interface is less than the shear strength. If the shear strength of the interface element is fully mobilized, which occurs when τ is equal to τ_f, then k_s is set equal to zero. When the normal stress, σ_n, is greater than or equal to zero, the value of τ_f is given by the relationship

$$\tau_f = c_i + \sigma_n \tan \delta$$

where

c_i = *cohesion intercept*
δ = angle of internal friction along the interface

and is shown in Figure 3a. When σ_n is less than zero, τ_f is computed using

$$\tau_f = \sigma_n\left(\frac{c_i}{\sigma_t}\right)$$

where
σ_t = tensile strength

Direct shear test results on soil-to-concrete interfaces and soil-to-steel interfaces by Potyondy (1961), Clough and Duncan (1969) and Peterson, Kulhawy, Nucci, and Wasil (1976) have shown that the value is proportional to the angle of internal friction of the soil. The value of the constant of proportionality is dependent upon both the type of soil and the type of material comprising the surface of the structure.

The direct shear tests performed by Clough and Duncan (1969) and Peterson, Kulhawy, Nucci, and Wasil (1976) have shown that for some materials, such as sand-to-concrete interfaces, the interface response during shear is nonlinear and dependent upon the normal stress. A nonlinear, stress-dependent hyperbolic curve is used to represent the relationship between shear stress and average relative shear displacement developing during primary loading of the interface (Figure 3a) and a linear shear stress-relative displacement response during unloading or reloading of the interface. The stress-dependent value of k_{st} is computed using the relationship

$$k_{st} = k_{si}(1 - R_{fi}\ SL_i)^2$$

where

k_{si} = the initial interface shear stiffness

R_{fi} = the failure ratio
SL_i = the stress level

The initial interface shear stiffness, k_{si}, is equal to

$$k_{si} = K_i \gamma_w \left(\frac{\sigma_n}{P_a}\right)^{n_i}$$

K_i = the interface modulus number
γ_w = unit weight of water
n_i = the interface modulus exponent
σ_n = normal stress
P_a = Atmospheric Pressure

The proportion of mobilized shear strength for each interface element is reflected in the value of the stress level, SL_i. SL_i is equal to the current shear stress, τ, divided by the shear stress at failure, τ_f.

$$SL_i = \frac{\tau}{\tau_f}$$

τ_f is computed using the equation above. SL_i ranges in value between zero and one.

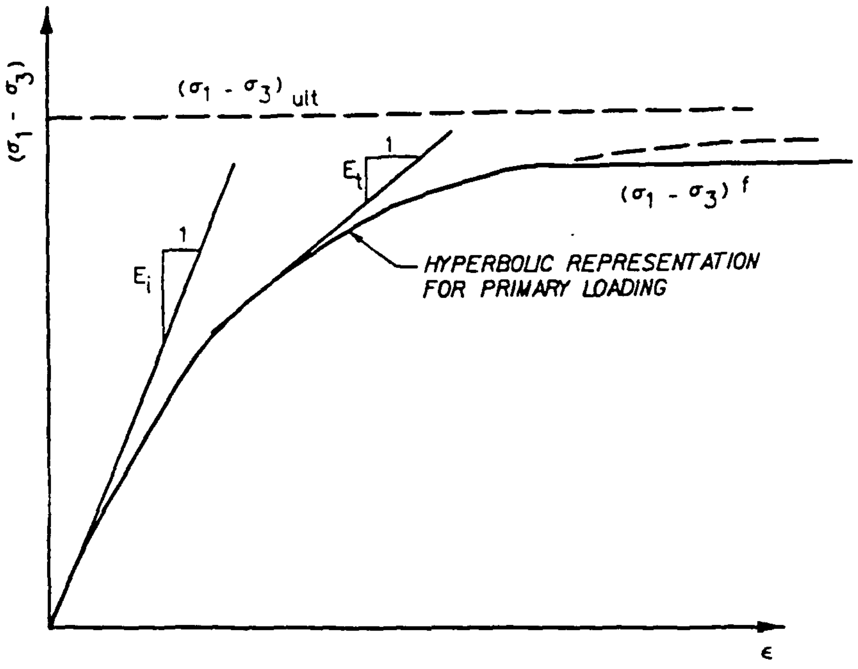
The failure ratio, R_{fi}, relates the ultimate shear stress, τ_{ult}, to the shear stress at failure.

$$\tau_f = R_{fi}\tau_{ult}$$

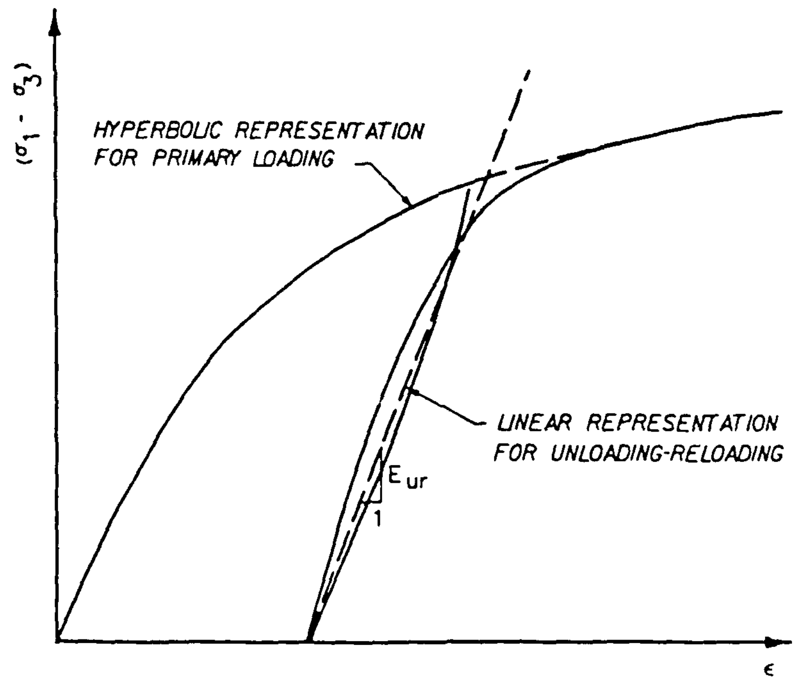
The ultimate shear stress is the asymptote to the shear stress-relative shear displacement hyperbola, as shown in Figure 4b. Direct shear tests on sand-to-concrete interfaces by Peterson, Kulhawy, Nucci, and Wasil (1976) have shown the value of R_{fi} typically ranges in value from 0.3 to 1.0.

The relationship between the average normal stress along the interface and the

tensile strength is shown in Figure 4b. The value of k_n is a constant value equal to 1 X 10^8 when σ_n is greater than or equal to σ_t. If σ_n is less than σ_t, then k_n is set equal to zero, assuring that additional tensile stresses do not accrue upon subsequent loadings. This procedure allows for separation to occur between two adjacent regions of the mesh along interface elements during the course of an incremental analysis.

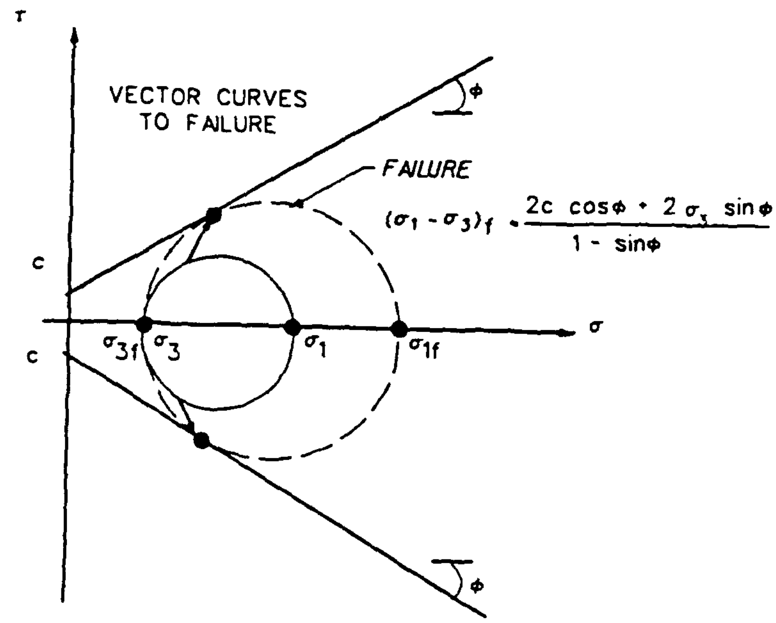


a. Hyperbolic representation of stress-strain curve for primary loading

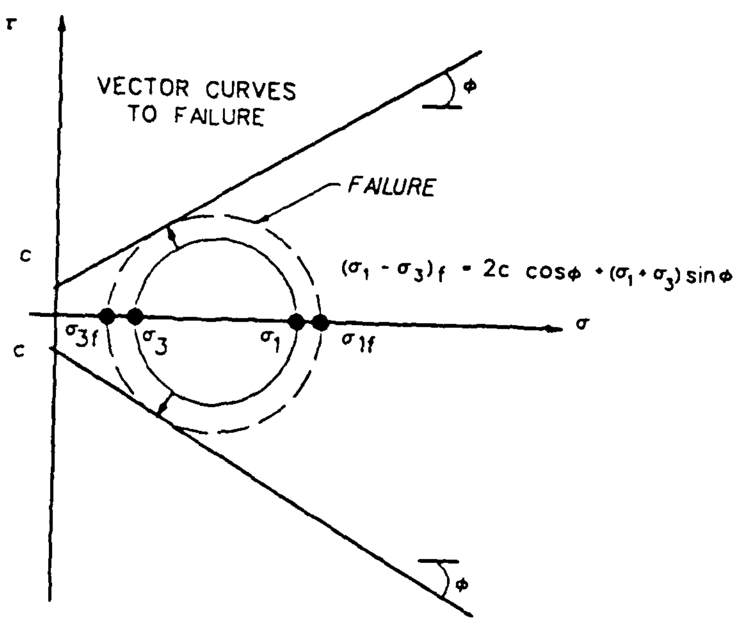


b. Linear unloading-reloading stress-strain relationship

Figure 1. Hyperbolic model for stress-strain behavior (after Duncan, Byrne, Wong, and Mabry 1978)

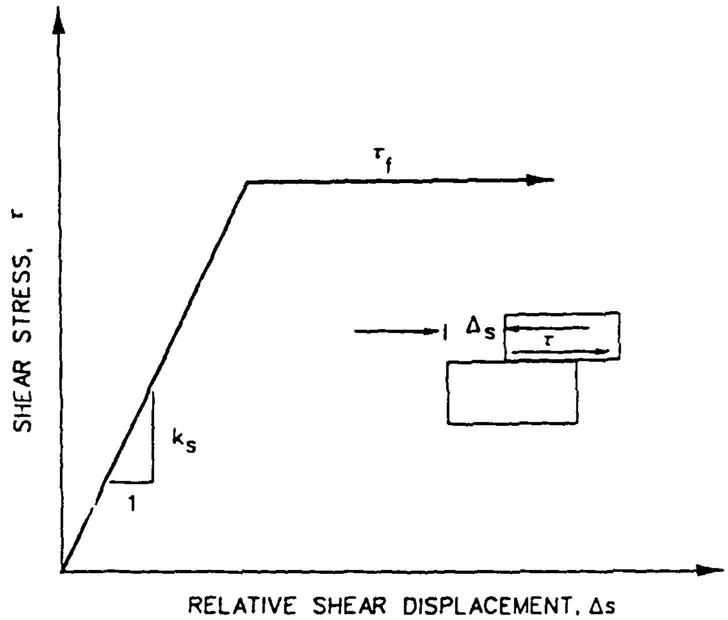


a. Duncan formulation, $\sigma_{3_f} = \sigma_3$

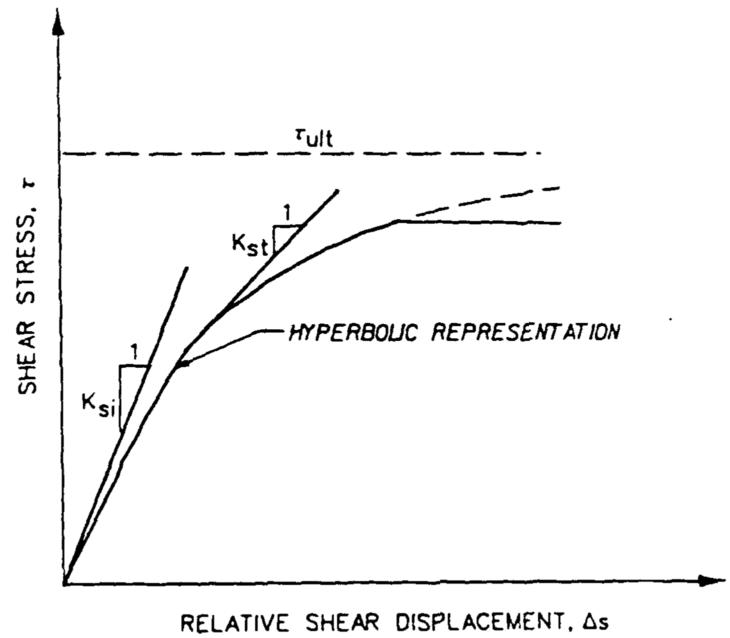


b. Peters formulation, $p = \frac{(\sigma_1 + \sigma_3)_f}{2} = \frac{(\sigma_1 + \sigma_3)}{2}$

Figure 2. Two procedures used to define deviator stress at failure, $(\sigma_1 - \sigma_3)_f$

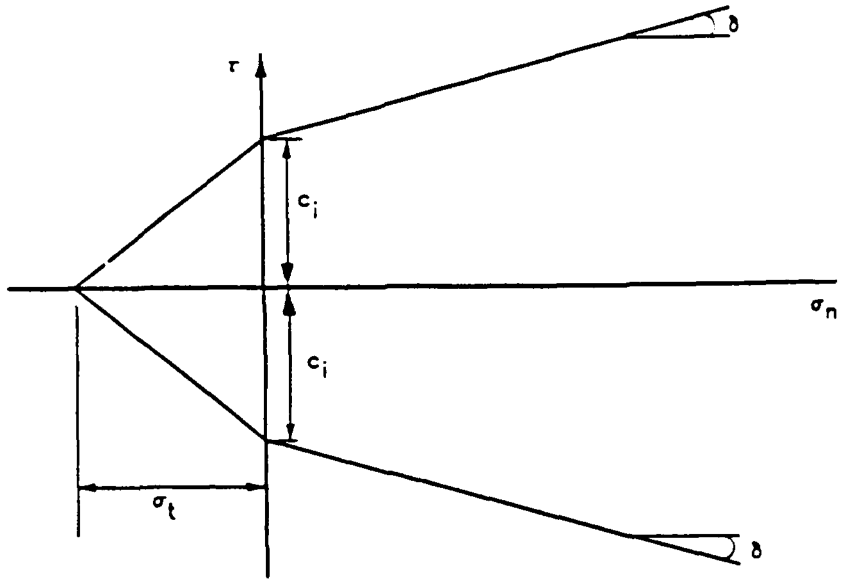


a. Bilinear stress-strain model representing interface behavior

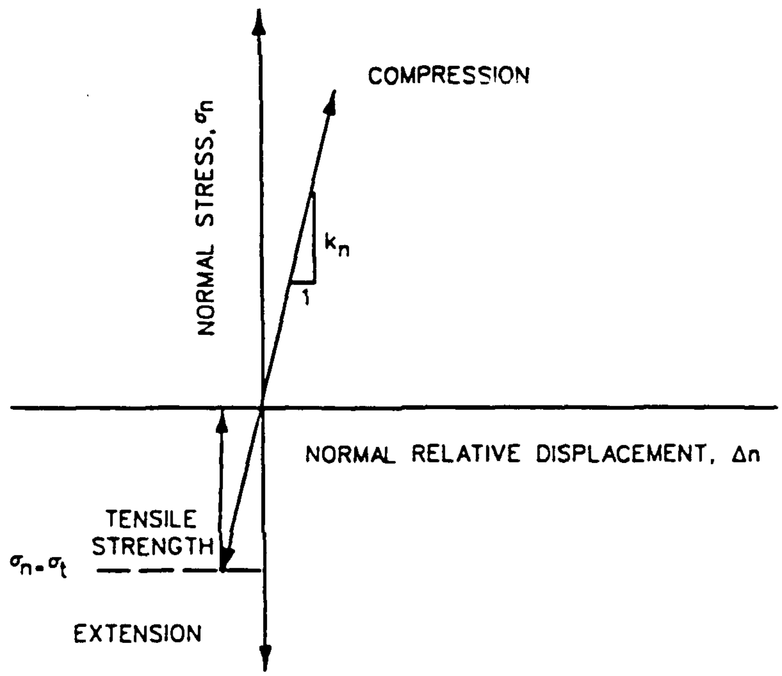


b. Hyperbolic representation of the variation of shear stress with relative shear displacement (k_{st} = tangent interface shear stiffness, k_{si} = initial interface shear stiffness)

Figure 3. Bilinear and hyperbolic models for interface shear stress-relative shear displacement behavior after Clough and Duncan (1969)



a. Strength criteria



b. Normal stress-normal relative displacement behavior

Figure 4. Interface strength criteria and normal stress-normal relative displacement relationship

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APPENDIX D: REANALYSIS OF TRIAXIAL TESTS

Computation of Hyperbolic Parameters

Sample: TP-84-6
Depth: 2-4'
Type: STD. Compaction
Descrip.: Glacial Till, ML

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	56.9	39.8	1.9	4.774E-04	54.1	3.2	5.915E-04	1.02	8.778E-03	0.4995	219.01
30	110.5	77.4	2.3	2.972E-04	105	4.1	3.905E-04	2.04	5.184E-03	0.5729	382.35
60	206.9	144.8	2.8	1.934E-04	196.6	4.8	2.442E-04	4.08	2.539E-03	0.5253	556.33
									Avg. Rf =	0.5325	

Soil Properties:
Phi = 38.7 deg
c = 300 psf

Computed Deviatoric Stress at Failure:

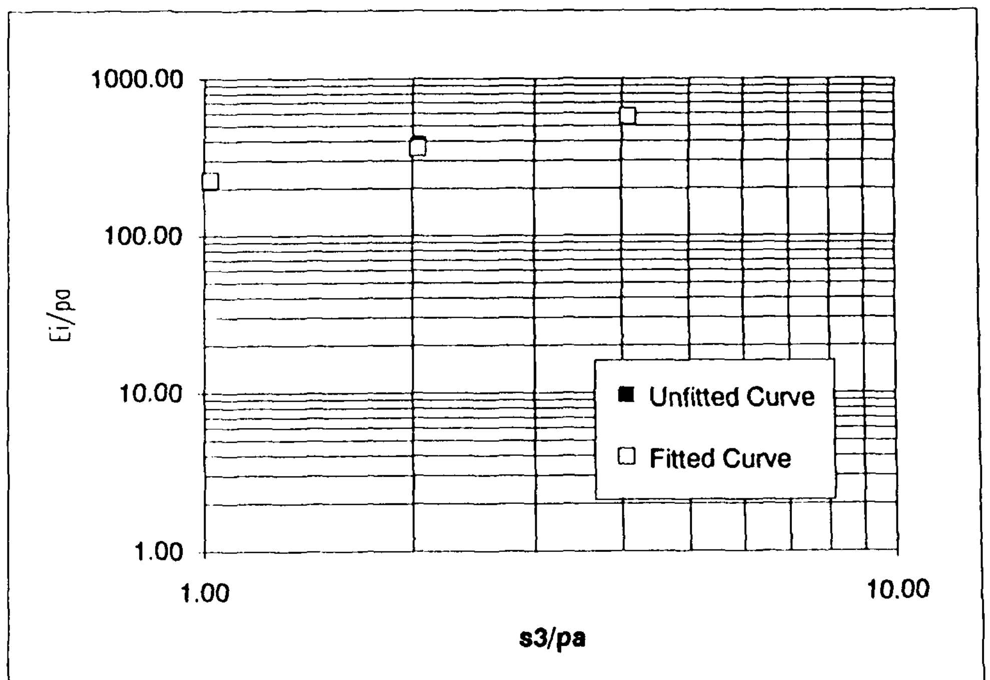
s3	(s1-s3)f
15	58.73
30	108.78
60	208.88

Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	2.34046674	5.47778455	0.02053508	7.6982E-05
0.30980392	2.58246466	6.6691237	0.80005767	0.09597847
0.61083392	2.74533571	7.53686815	1.67694416	0.37311807
Summations:				
0.92941176	7.6682671	19.6837764	2.49753691	0.46917352

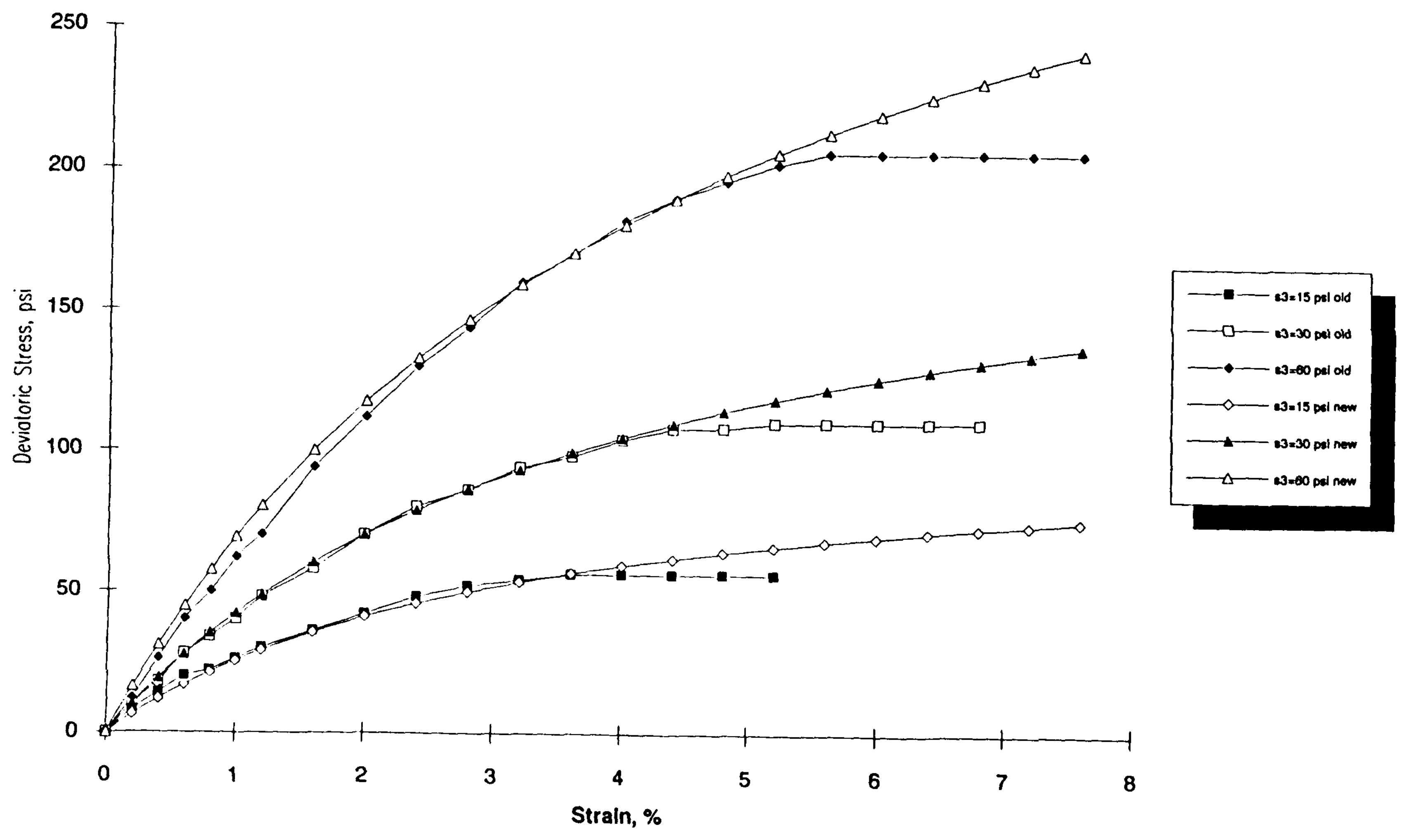
Hyperbolic Parameters:
n = 0.6724728
k = 222.71749

New Curve Values:

x	y
1.020408	225.763926
2.040816	359.823094
4.081633	573.486922



Test and Compute:d Stress-Strain Curves



Computation of Hyperbolic Parameters

Sample: TP-84-6
Depth: 2-4'
Type: Mod. Compaction
Descrip.: Glacial Till, ML

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	95.4	66.8	0.48	7.186E-05	90.6	0.76	8.389E-05	1.02	4.296E-03	0.4098	1327.74
30	156.8	109.8	0.68	6.193E-05	149	1.03	6.913E-05	2.04	2.056E-03	0.3224	1418.75
60	246.7	172.7	0.73	4.227E-05	234.4	1.16	4.949E-05	4.08	1.679E-03	0.4141	2266.39
									Avg. Rf =	0.3821	

Soil Properties:
Phi = 38.8 deg
c = 1740 psf

Computed Deviatoric Stress at Failure:

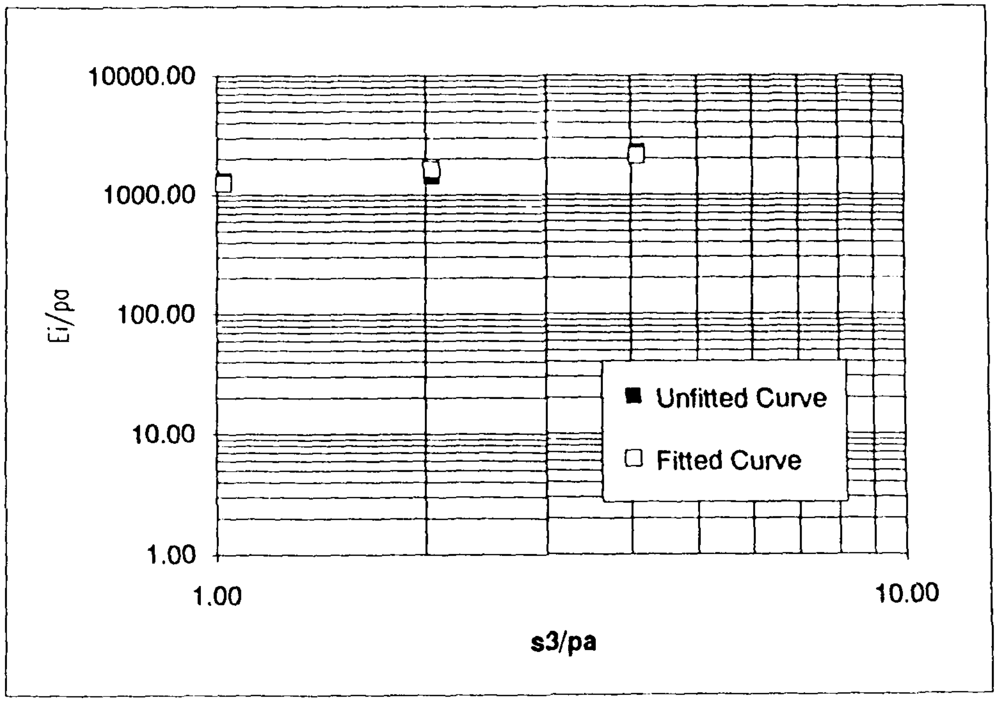
s3	(s1-s3)f
15	100.78
30	151.13
60	251.81

Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	3.12311357	9.75383837	0.02740196	7.6982E-05
0.30980392	3.15190711	9.93451845	0.97647318	0.09597847
0.61083392	3.35533505	11.2582733	2.04955245	0.37311807
Summations:				
0.92941176	9.63035574	30.9466301	3.05342759	0.46917352

Hyperbolic Parameters:
n = 0.38571153
k = 1232.03667

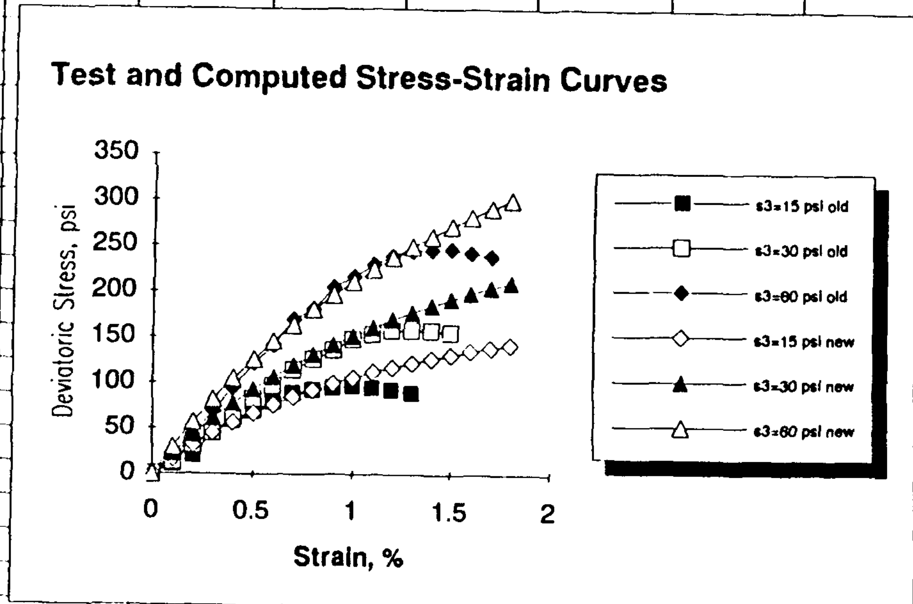
New Curve Values:

x	y
1.020408	1241.67472
2.040816	1622.25297
4.081633	2119.47999

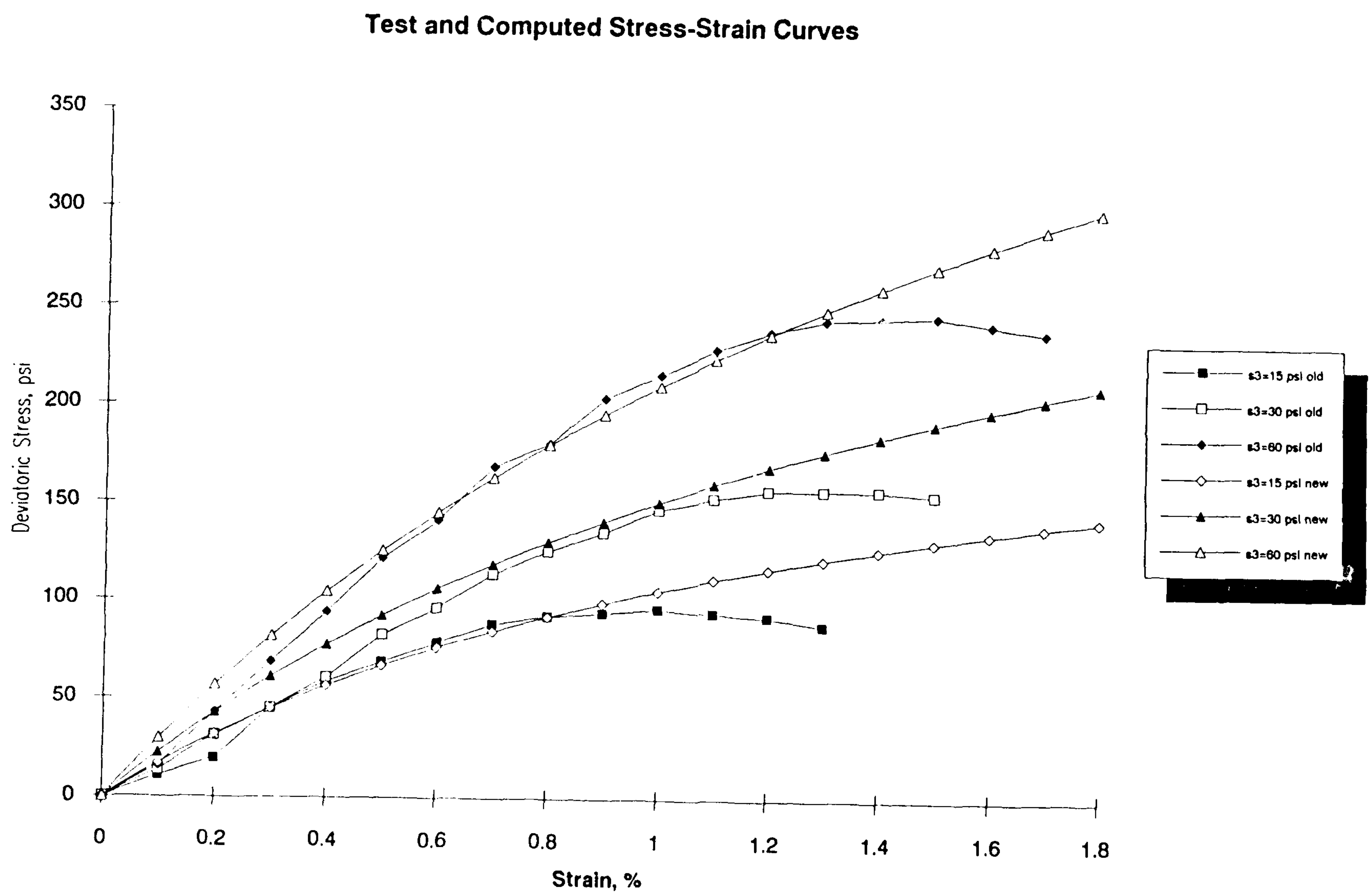


Compute Stress-Strain Curves					Sigma 3:	15	(s1-s3)f:	95.4	n=	0.385712
						30		156.8	k=	1232.037
						60		246.7	Rf=	0.382127

	Test Data			Compted Curves		
e	x1	x2	x3	x1	x2	x3
0	0	0	0	0.00	0.00	0.00
0.1	10.8	13.7	15.7	17.01	22.54	29.72
0.2	19.6	31.4	43.1	31.85	42.73	56.83
0.3	45.1	45.1	68.6	44.91	60.92	81.65
0.4	58.8	60.8	94.1	56.49	77.40	104.46
0.5	68.6	82.4	121.6	66.83	92.39	125.50
0.6	78.4	96.1	141.2	76.12	106.09	144.96
0.7	88.2	114	168.6	84.52	118.66	163.02
0.8	92.2	126	180.4	92.13	130.23	179.82
0.9	94.1	135	203.9	99.08	140.92	195.50
1	96.1	147	216	105.44	150.82	210.15
1.1	94.1	153	229	111.28	160.02	223.87
1.2	92.2	157	238	116.67	168.59	236.76
1.3	88.2	157	244	121.66	176.59	248.89
1.4		157	245	126.28	184.08	260.31
1.5		155	246	130.58	191.11	271.10
1.6			242	134.60	197.71	281.30
1.7			238	138.35	203.93	290.95
1.8				141.86	209.79	300.11



C846-2M.XLS Chart 1



Computation of Hyperbolic Parameters

Sample: TP-84-6
Depth: 8-10'
Type: STD. Compaction
Descrip.: Glacial Till, SM

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	61.7	43.2	0.62	1.435E-04	58.6	1.15	1.962E-04	1.02	9.949E-03	0.6138	831.25
30	110.3	77.2	0.77	9.974E-05	104.8	1.37	1.307E-04	2.04	5.164E-03	0.5696	1134.21
60	227.2	159	1.79	1.126E-04	215.8	3.42	1.585E-04	4.08	2.816E-03	0.6398	1094.19
									Avg. Rf =	0.6077	

Soil Properties:
Phi = 40.4 deg
c = 110 psf

Computed Deviatoric Stress at Failure:

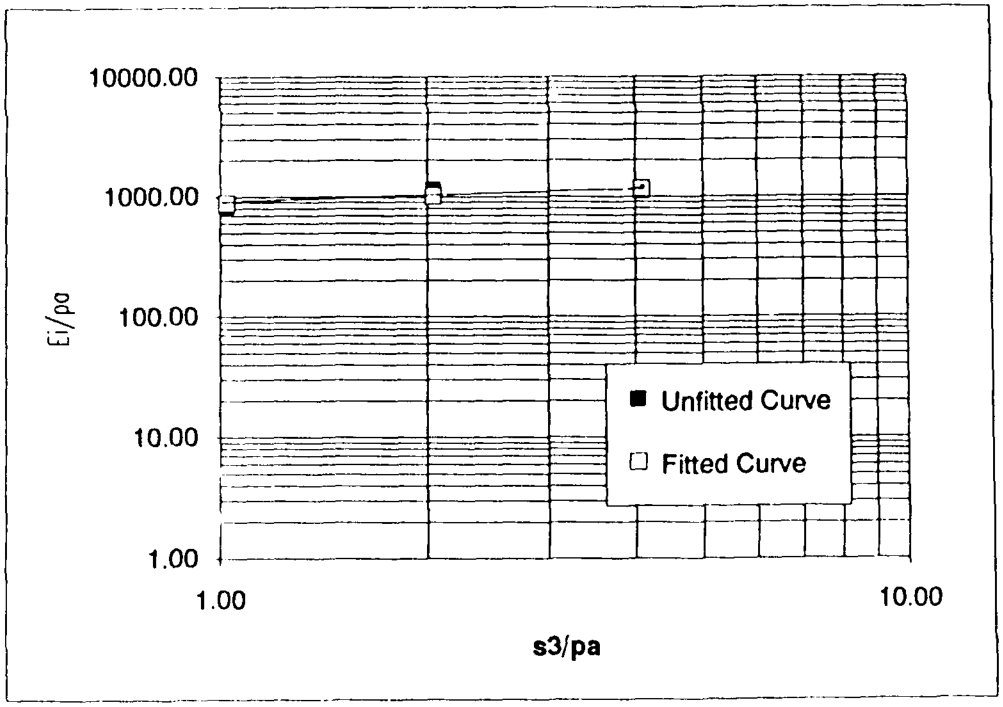
s3	(s1-s3)f
15	58.56
30	113.82
60	224.33

Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	2.91972965	8.52482121	0.02561749	7.6982E-05
0.30980392	3.05469212	9.33114397	0.94635559	0.09597847
0.61083392	3.03909136	9.2360763	1.85638008	0.37311807
Summations:				
0.92941176	9.01351313	27.0920415	2.82835316	0.46917352

Hyperbolic Parameters:
n = 0.19825552
k = 877.170548

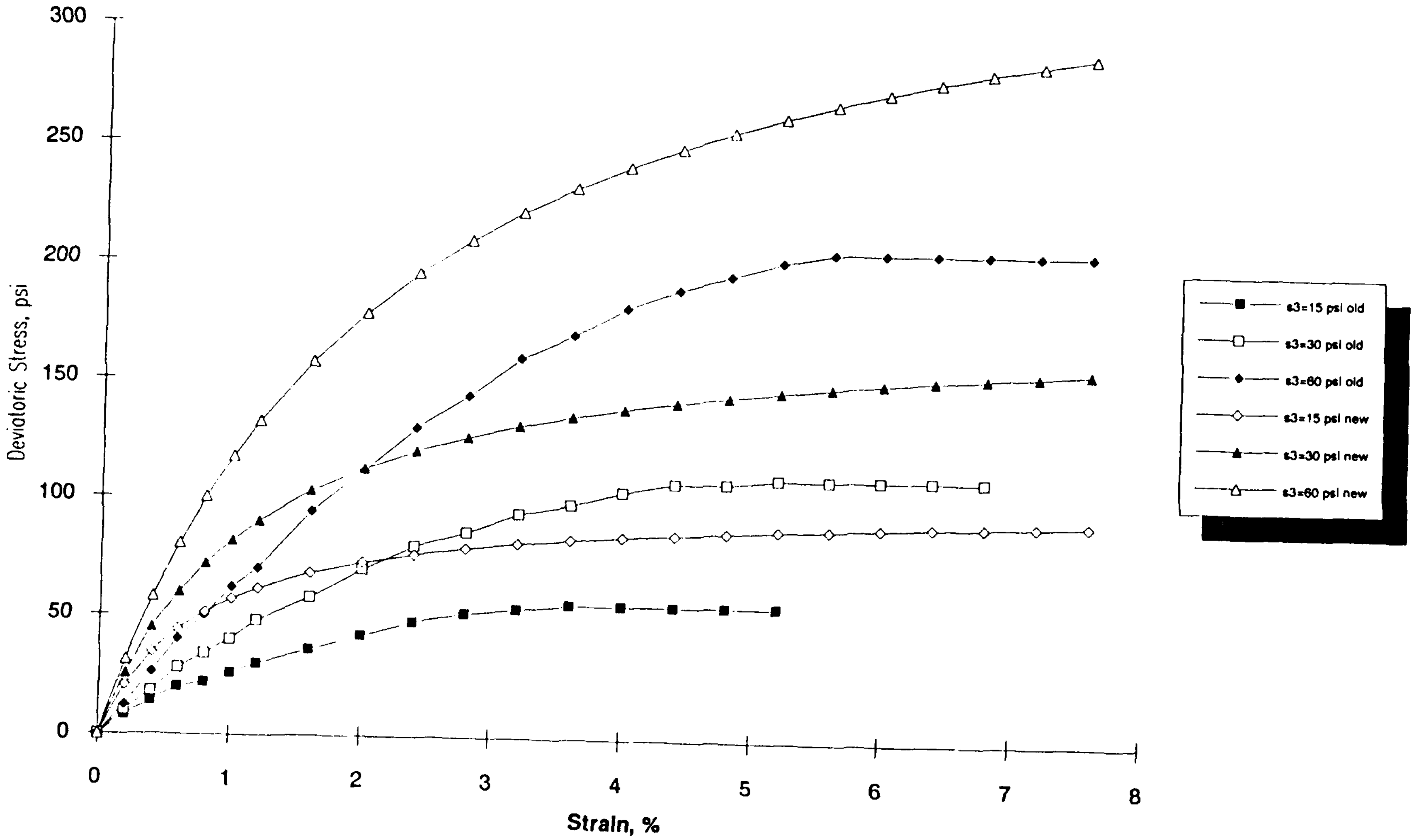
New Curve Values:

x	y
1.020408	880.690923
2.040816	1010.42569
4.081633	1159.2717



C846-8S.XLS Chart 1

Test and Computed Stress-Strain Curves



W846-8S.XLS

Adjusted

Computation of Hyperbolic Parameters

Sample: TP-84-6
Depth: 8-10'
Type: STD. Compaction
Descrip.: Glacial Till, SM

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	61.7	43.2	0.62	1.435E-04	58.6	1.15	1.962E-04	1.02	9.949E-03	0.6138	831.25
30	110.3	77.2	0.77	9.974E-05	104.8	1.37	1.307E-04	2.04	5.164E-03	0.5696	1134.21
60	227.2	159	1.79	1.126E-04	215.8	3.42	1.585E-04	4.08	2.816E-03	0.6398	1094.19
									Avg. Rf =	0.6500	

Soil Properties:
Phi = 40.4 deg
c = 110 psf

Computed Deviatoric Stress at Failure:

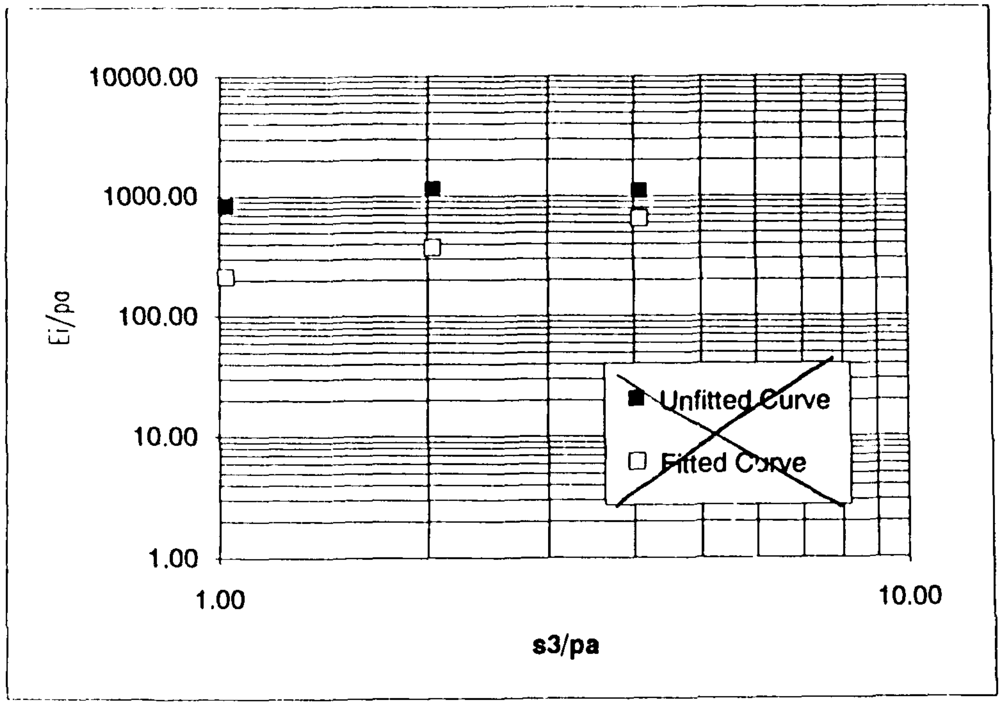
s3	(s1-s3)f
15	58.56
30	113.82
60	224.33

Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	2.91972965	8.52482121	0.02561749	7.6982E-05
0.30980392	3.05469212	9.33114397	0.94635559	0.09597847
0.61083392	3.03909136	9.2360763	1.85638008	0.37311807
Summations:				
0.92941176	9.01351313	27.0920415	2.82835316	0.46917352

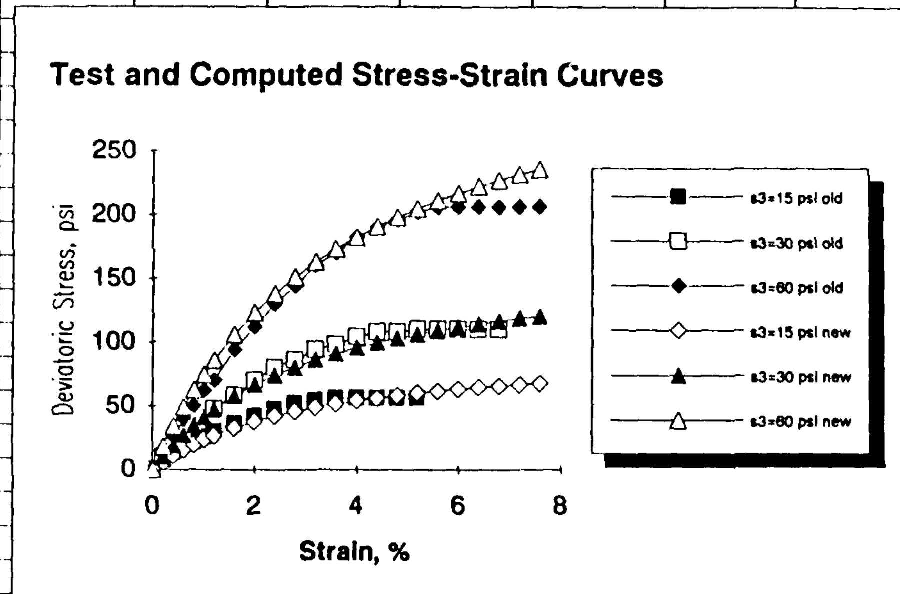
Hyperbolic Parameters:
n = 0.8
k = 210

New Curve Values:

x	y
1.020408	213.421631
2.040816	371.588642
4.081633	646.973403



Compute Stress-Strain Curves					Sigma 3:	15	(s1-s3)f:	61.7	n=	0.8
						30		110.3	k=	210
						60		227.2	Rf=	0.65
	Test Data			Compted Curves						
e	x1	x2	x3	x1	x2	x3				
0	0	0	0	0.00	0.00	0.00				
0.2	8	10	12	5.89	10.26	18.04				
0.4	14	18	26	11.08	19.36	34.31				
0.6	20	28	40	15.71	27.47	49.05				
0.8	22	34	50	19.85	34.75	62.48				
1	26	40	62	23.58	41.32	74.76				
1.2	30	48	70	26.96	47.28	86.04				
1.6	36	58	94	32.83	57.69	106.02				
2	42	70	112	37.78	66.46	123.18				
2.4	48	80	130	41.99	73.96	138.08				
2.8	52	86	144	45.62	80.44	151.15				
3.2	54	94	160	48.79	86.10	162.69				
3.6	56	98	170	51.58	91.09	172.96				
4	56	104	182	54.04	95.51	182.16				
4.4	56	108	190	56.25	99.47	190.45				
4.8	56	108	196	58.22	103.02	197.96				
5.2	56	110	202	60.01	106.23	204.79				
5.6		110	206	61.63	109.14	211.04				
6		110	206	63.10	111.80	216.76				
6.4		110	206	64.45	114.24	222.03				
6.8		110	206	65.69	116.48	226.90				
7.2			206	66.84	118.54	231.41				
7.6			206	67.89	120.46	235.60				



Computation of Hyperbolic Parameters

Sample: TP-84-6
Depth: 8-10'
Type: Mod. Compaction
Descrip.: Glacial Till, SM

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	112.9	79	0.82	1.038E-04	107.3	1.19	1.109E-04	1.02	1.921E-03	0.2168	772.62
30	161.6	113.1	0.7	6.189E-05	153.5	1.07	6.971E-05	2.04	2.112E-03	0.3413	1444.08
60	292.2	204.5	0.92	4.499E-05	277.6	1.42	5.115E-05	4.08	1.233E-03	0.3603	2021.96
									Avg. Rf =	0.3061	

Soil Properties:
Phi = 41.7 deg
c = 1540 psf

Computed Deviatoric Stress at Failure:

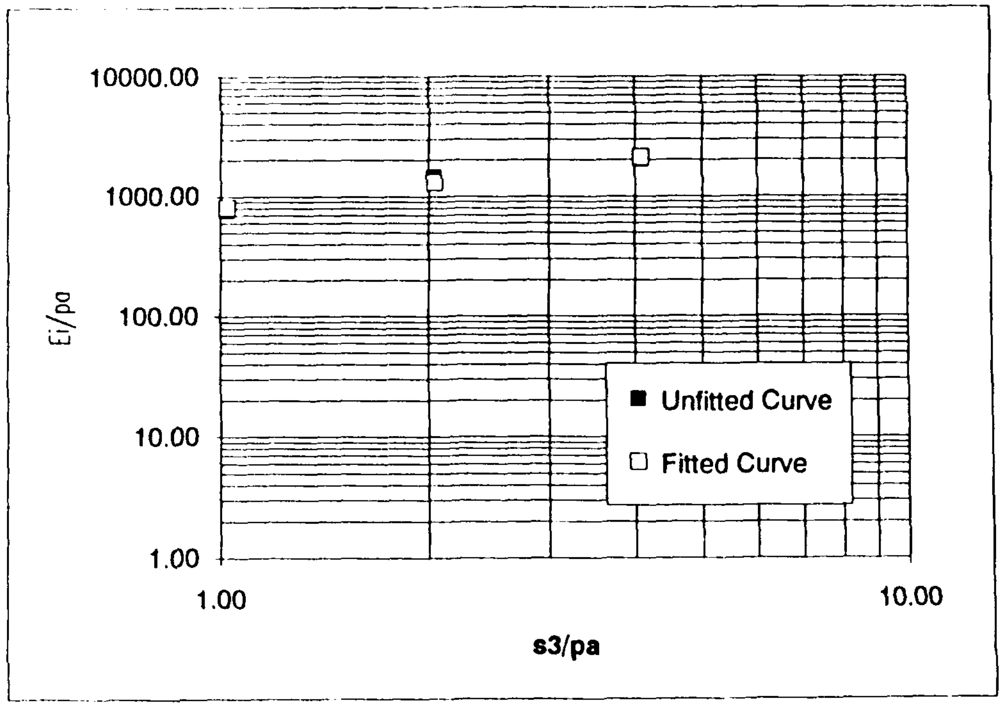
s3	(s1-s3)f
15	107.32
30	166.93
60	286.16

Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	2.88796395	8.3403358	0.02533878	7.6982E-05
0.30980392	3.15959219	9.98302281	0.97885405	0.09597847
0.61083392	3.3057719	10.9281278	2.01927759	0.37311807
Summations:				
0.92941176	9.35332804	29.2514864	3.02347041	0.46917352

Hyperbolic Parameters:
n = 0.69396397
k = 799.435178

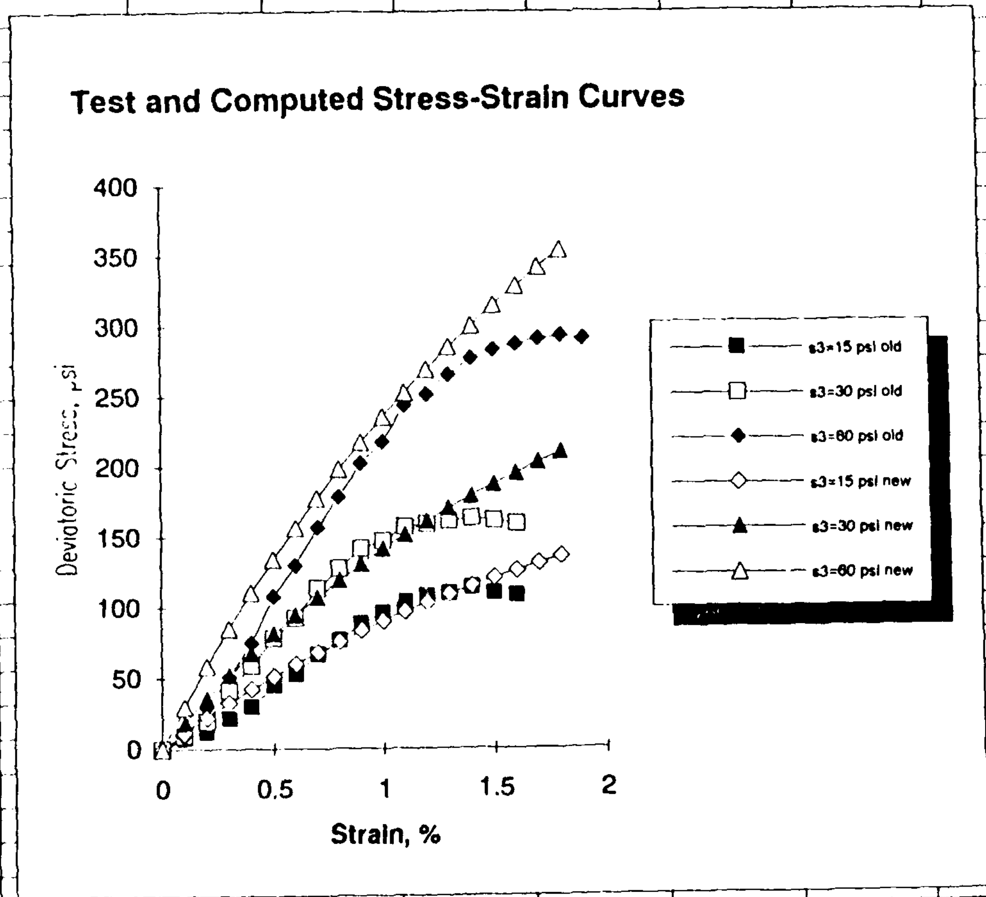
New Curve Values:

x	y
1.020408	810.722157
2.040816	1311.52331
4.081633	2121.6805



Compute Stress-Strain Curves | **Sigma 3:** 15, 30, 60 | **(s1-s3)f:** 112.9, 161.6, 292.2 | **n=** 0.693964 | **k=** 799.4352 | **Rf=** 0.306146

	Test Data			Compted Curves		
e	x1	x2	x3	x1	x2	x3
0	0	0	0	0.00	0.00	0.00
0.1	7.8	9.8	9.8	11.54	18.60	30.20
0.2	11.8	19.6	29.4	22.39	35.93	58.55
0.3	21.6	41.2	51	32.59	52.13	85.21
0.4	29.4	58.8	74.5	42.21	67.29	110.33
0.5	45.1	78.4	107.8	51.30	81.51	134.04
0.6	52.9	92.2	129.4	59.89	94.88	156.46
0.7	66.7	113.7	156.9	68.03	107.48	177.68
0.8	76.5	127.5	178.4	75.76	119.36	197.80
0.9	88.2	141.2	202	83.09	130.59	216.91
1	96.1	147.1	217.6	90.07	141.22	235.07
1.1	103.9	156.9	243.1	96.71	151.29	252.36
1.2	107.8	158.8	251	103.05	160.85	268.84
1.3	109.8	160.8	264.7	109.10	169.94	284.57
1.4	113.7	162.7	276.5	114.87	178.59	299.59
1.5	109.8	160.8	282.4	120.40	186.83	313.95
1.6	107.8	158.8	286.3	125.69	194.69	327.69
1.7			290.2	130.76	202.20	340.86
1.8			292.2	135.62	209.38	353.48
1.9			290.2			



Computation of Hyperbolic Parameters

Sample: TP-84-5
Depth: 8.5-9.5'
Type: STD. Compaction
Descrip.: Glacial Till, CL

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	44.9	31.4	1.65	5.255E-04	42.7	4.28	1.002E-03	1.02	1.813E-02	0.8141	300.60
30	89.6	62.7	2.27	3.620E-04	85.1	5.42	6.369E-04	2.04	8.726E-03	0.7818	414.87
60	167.4	117.2	3.95	3.370E-04	159	8	5.031E-04	4.08	4.102E-03	0.6866	388.69
									Avg. Rf =	0.7608	

Soil Properties:
Phi = 35.2 deg
c = 225 psf

Computed Deviatoric Stress at Failure:

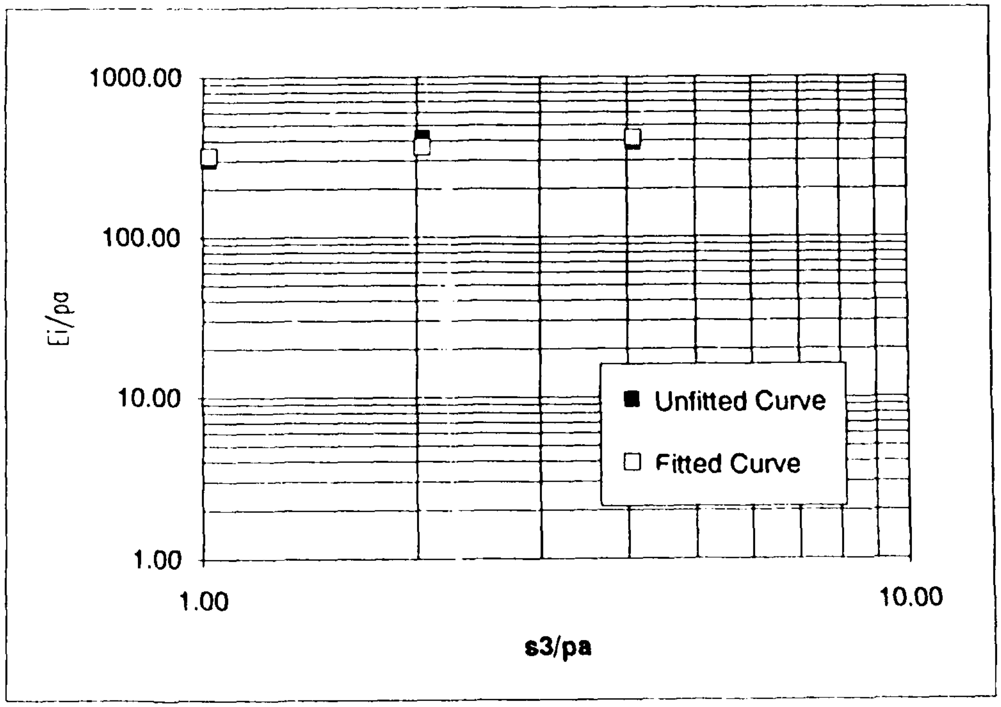
s3	(s1-s3)f
15	46.86
30	87.68
60	169.34

Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	2.47798978	6.14043335	0.02174169	7.6982E-05
0.30980392	2.61791716	6.85349025	0.811041	0.09597847
0.61083392	2.58959906	6.70602329	1.58181493	0.37311807
Summations:				
0.92941176	7.685506	19.6999469	2.41459763	0.46917352

Hyperbolic Parameters:
n = 0.185379
k = 319.450955

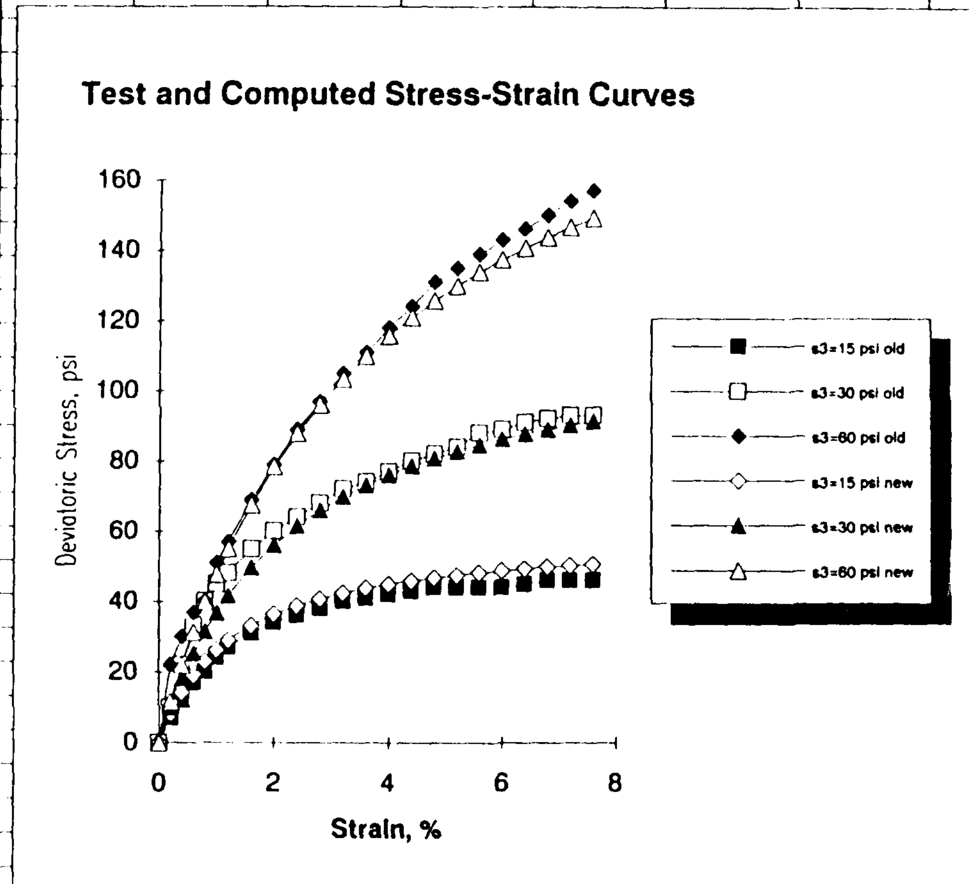
New Curve Values:

x	y
1.020408	320.649593
2.040816	364.615673
4.081633	414.610191



Compute Stress-Strain Curves					Sigma 3:	15	(s1-s3)f:	44.9	n=	0.185379
						30		89.6	k=	319.451
						60		167.4	Rf=	0.760844

	Test Data			Compted Curves		
e	x1	x2	x3	x1	x2	x3
0	0	0	0	0.00	0.00	0.00
0.2	7	10	22	8.13	9.83	11.55
0.4	12	22	30	14.29	18.14	21.95
0.6	17	33	37	19.12	25.26	31.36
0.8	20	40	40	23.01	31.43	39.91
1	24	45	51	26.20	36.83	47.73
1.2	27	48	57	28.88	41.60	54.89
1.6	31	55	69	33.11	49.62	67.57
2	34	60	79	36.29	56.12	78.44
2.4	36	64	89	38.78	61.48	87.86
2.8	38	68	97	40.78	65.99	96.11
3.2	40	72	105	42.42	69.82	103.39
3.6	41	74	111	43.79	73.13	109.86
4	42	77	118	44.95	76.01	115.65
4.4	43	80	124	45.94	78.54	120.86
4.8	44	82	131	46.81	80.79	125.58
5.2	44	84	135	47.56	82.78	129.86
5.6	44	88	139	48.23	84.58	133.78
6	44	89	143	48.83	86.20	137.37
6.4	45	91	146	49.36	87.67	140.67
6.8	46	92	150	49.84	89.01	143.72
7.2	46	93	154	50.27	90.23	146.54
7.6	46	93	157	50.67	91.35	149.17
8	46	94	159	51.03	92.39	151.61



Computation of Hyperbolic Parameters

Sample: TP-84-5
Depth: 8.5-9.5'
Type: MOD. Compaction
Descrip.: Glacial Till, CL

pa = 14.7 psi

		Data for Deviatoric Modulus Parameters									
		70% Stress Level			95% Stress Level						
s3 psi	(s1-s3)f psi	(s1-s3) psi	ea %	ea/(s1-s3)	(s1-s3) psi	ea %	ea/(s1-s3)	s3/pa	1/(s1-s3)ult	Rf	Ei/pa
15	91.2	63.8	0.84	1.317E-04	86.6	1.34	1.547E-04	1.02	4.615E-03	0.4209	732.27
30	142.7	99.9	0.89	8.909E-05	135.6	1.54	1.136E-04	2.04	3.766E-03	0.5374	1224.17
60	248.5	174	1.11	6.379E-05	236.1	2.09	8.852E-05	4.08	2.523E-03	0.6270	1901.05
									Avg. Rf =	0.5284	

Soil Properties:
Phi = 39.5 deg
c = 1300 psf

Computed Deviatoric Stress at Failure:

s3	(s1-s3)f
15	90.72
30	143.15
60	248.02

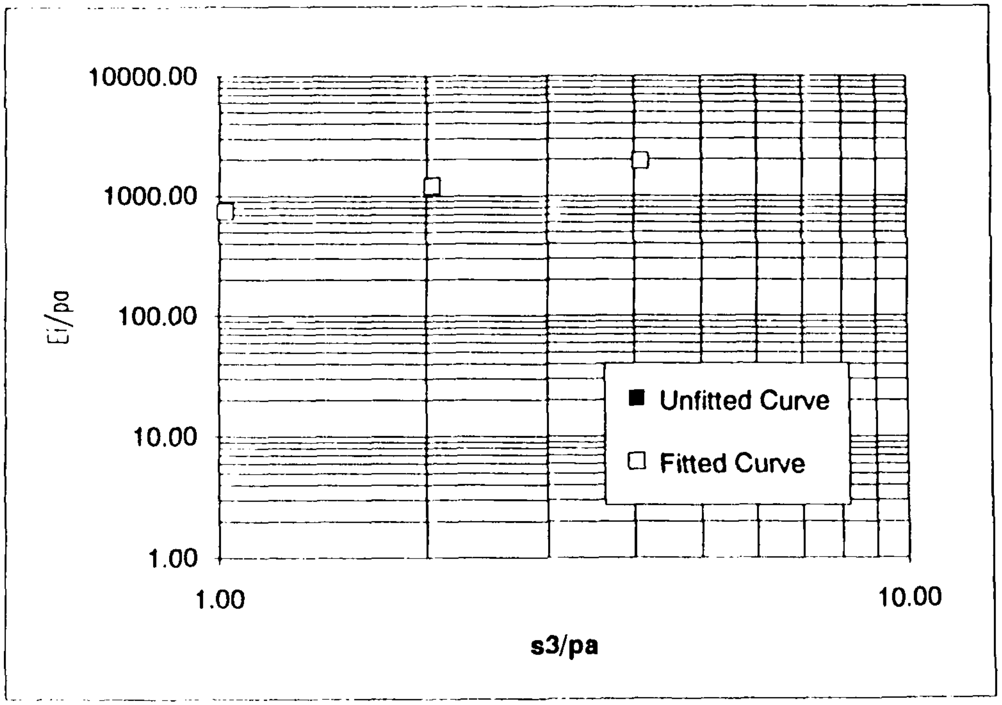
Compute Least Square Fit of Data:				
logx	logy	(logy)^2	logylogx	(logx)^2
0.00877392	2.86467231	8.20634744	0.02513442	7.6982E-05
0.30980392	3.08784226	9.5347698	0.95662564	0.09597847
0.61083392	3.27899311	10.7517958	2.0029202	0.37311807

Summations:
0.92941176 9.23150768 28.4929131 2.98468025 0.46917352

Hyperbolic Parameters:
n = 0.68817196
k = 731.09002

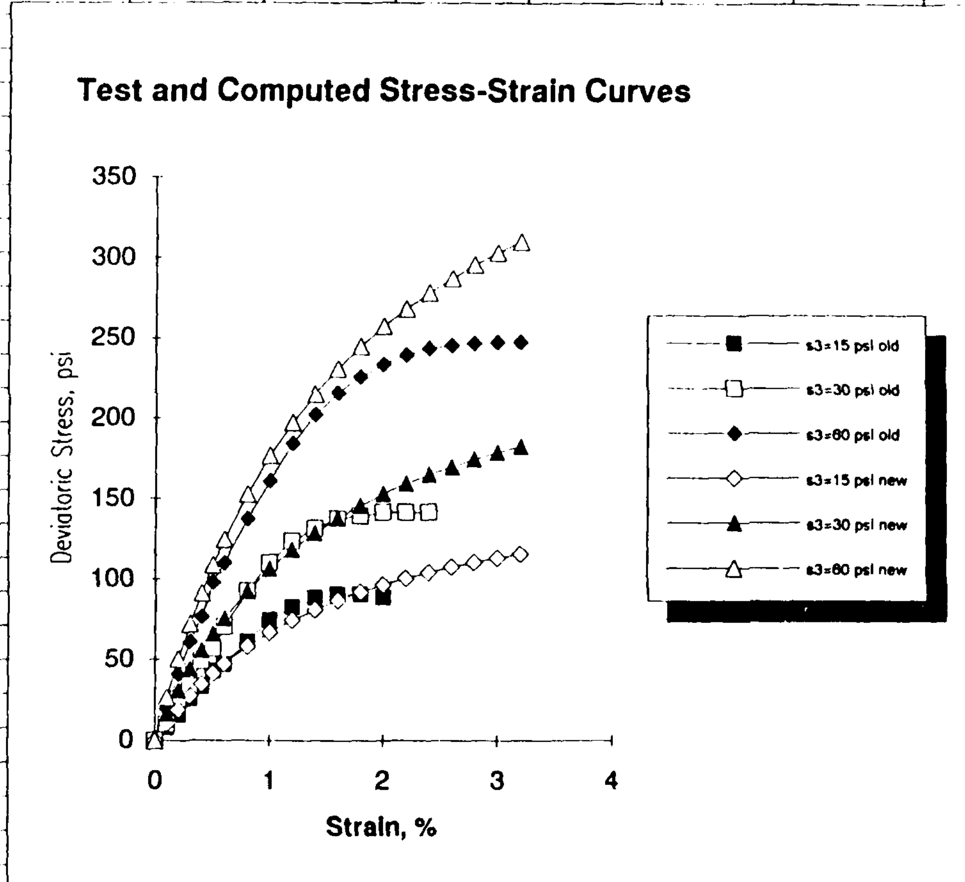
New Curve Values:

x	y
1.020408	741.325304
2.040816	1194.45344
4.081633	1924.5519



Compute Stress-Strain Curves				Sigma 3:		15	(s1-s3)f:	91.2	n=	0.688172
						30		142.7	k=	731.09
						60		248.5	Rf=	0.528445

	Test Data			Compted Curves		
e	x1	x2	x3	x1	x2	x3
0	0	0	0	0.00	0.00	0.00
0.1	7.8	9.8	19.6	10.25	16.49	26.69
0.2	15.7	25.5	41.2	19.35	31.08	50.50
0.3	25.5	33.3	60.8	27.49	44.08	71.90
0.4	33.3	49	76.5	34.80	55.74	91.21
0.5	43.1	56.9	98	41.41	66.25	108.74
0.6	47.1	70.6	109.8	47.42	75.78	124.72
0.8	60.8	92.2	137.3	57.92	92.40	152.79
1	74.5	109.8	160.8	66.80	106.40	176.64
1.2	82.4	123.5	184.3	74.40	118.35	197.16
1.4	88.2	131.4	202	80.98	128.68	214.99
1.6	90.2	137.3	215.7	86.73	137.69	230.64
1.8	90.2	139.2	225.5	91.81	145.62	244.48
2	88.2	141.2	233.3	96.32	152.65	256.81
2.2		141.2	239.2	100.35	158.93	267.87
2.4		141.2	243.1	103.97	164.58	277.83
2.6			245.1	107.25	169.67	286.86
2.8			246.1	110.23	174.30	295.08
3			247.1	112.95	178.52	302.59
3.2			247.1	115.45	182.38	309.49



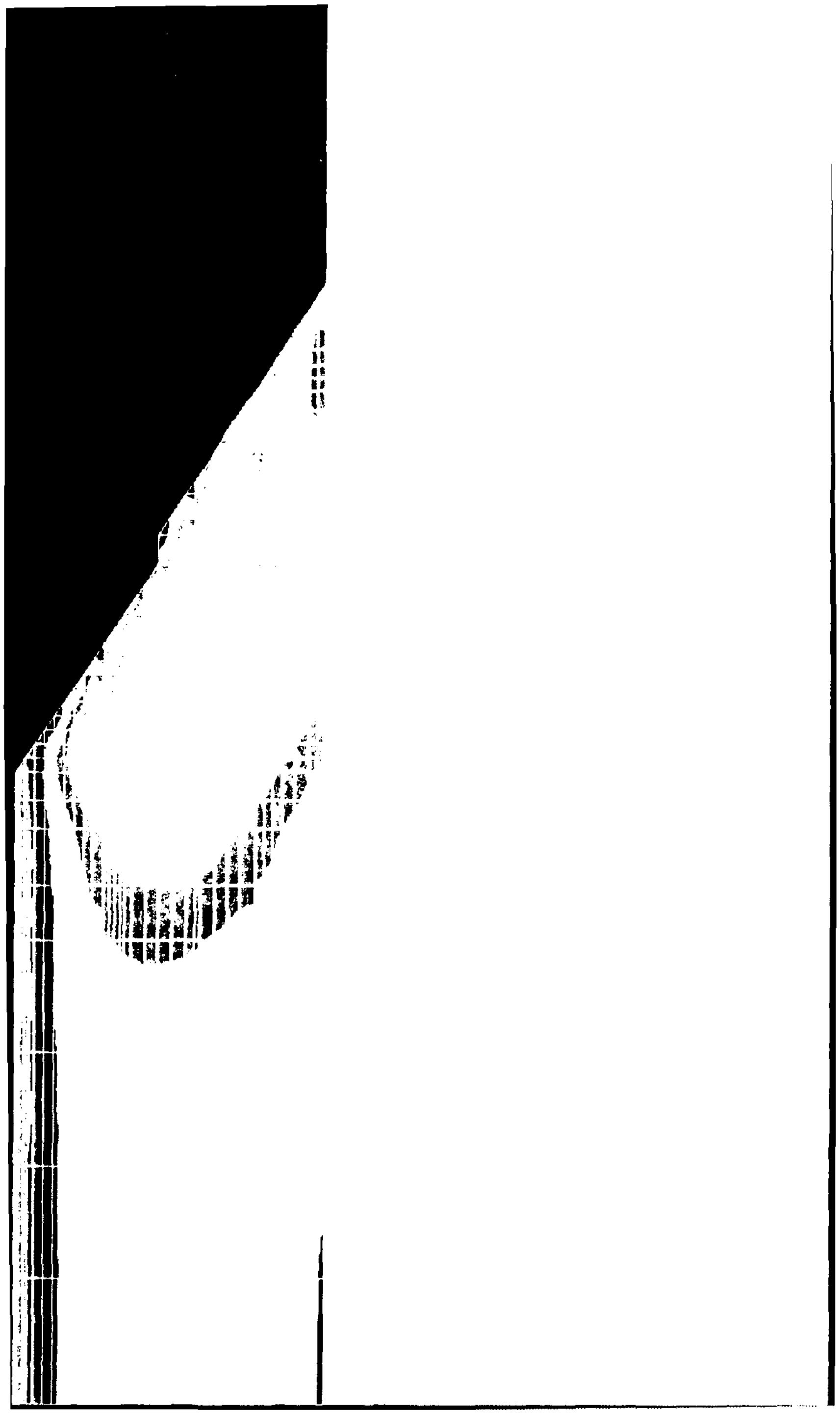
SUBJECT: Hyperbolic Parameters	COMPUTED BY:	DATE:	FILE NO.
	CHECKED BY:	DATE:	SHEET NO.

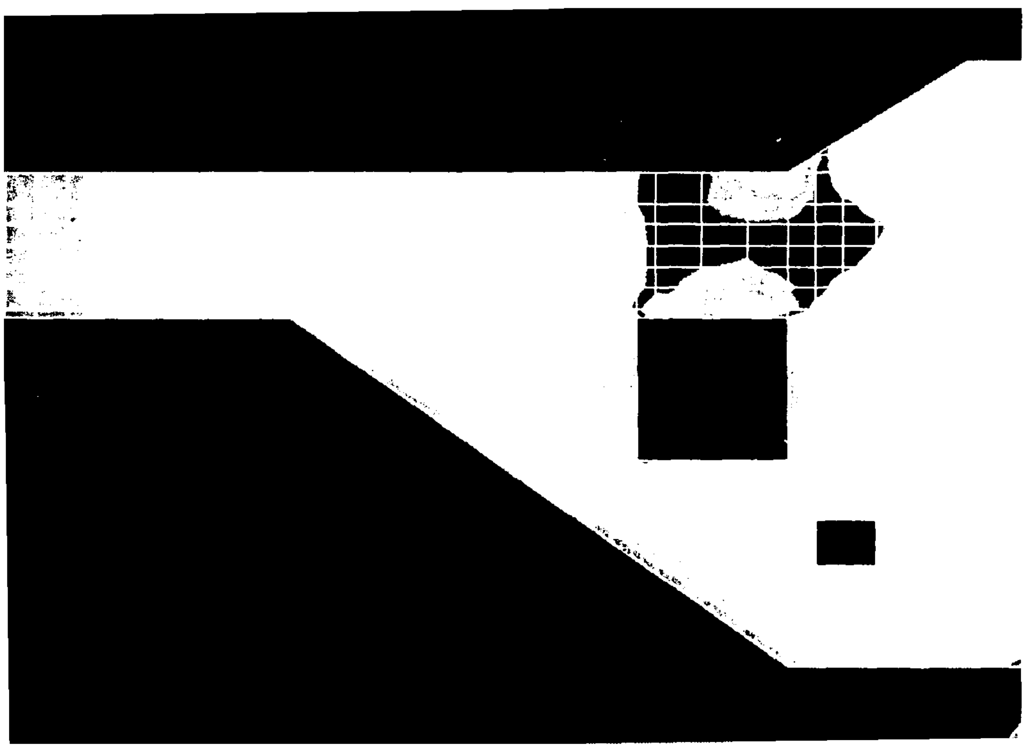
Summary

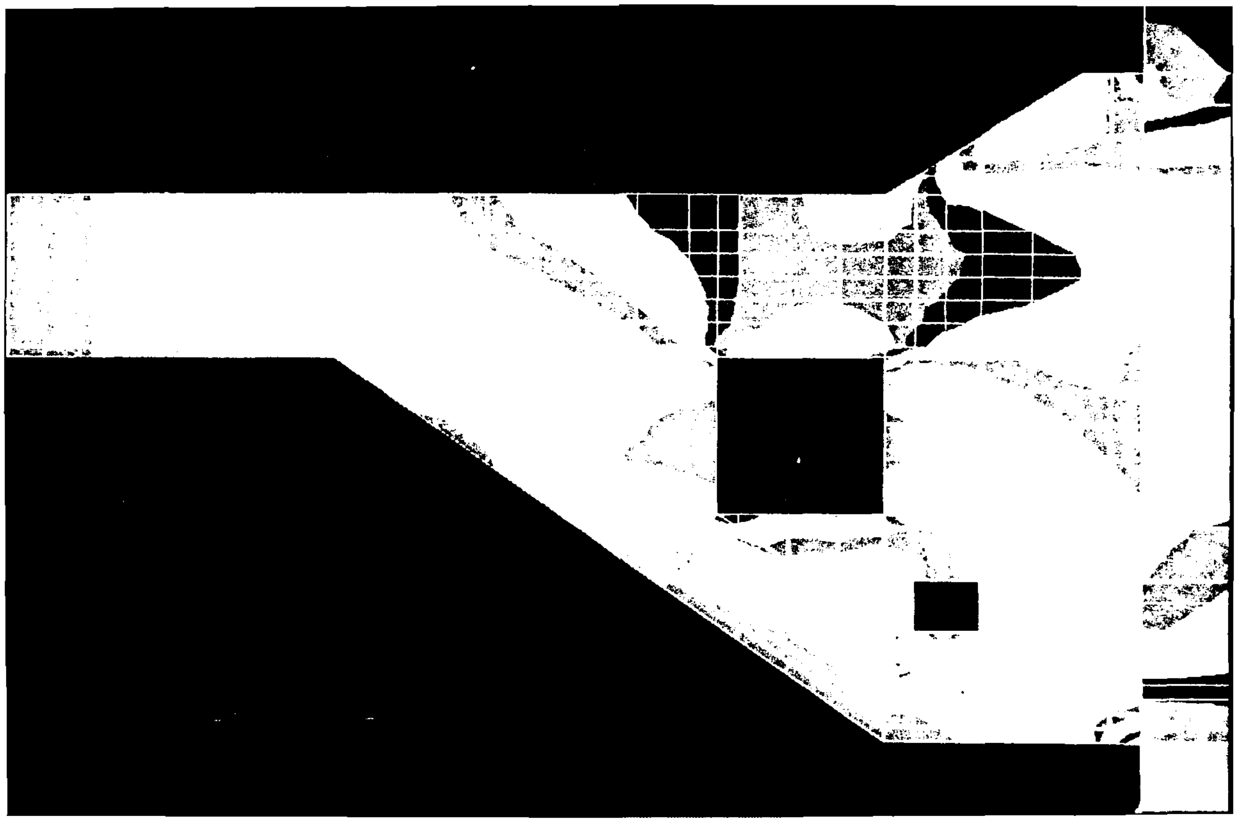
Test	R_f	n	K	ϕ	c psf
84-6-2S, ML	0.53	0.67	223	38.7	300
84-6-2M, ML	0.38	0.39	1232	38.8	1740
84-6-8S, SM	0.61	0.20	877	40.4	110
adjusted ?	0.30	0.65	210		
84-6-8M, SM	0.31	0.69	799	41.7	1540
84-5-2S, CL	0.76	0.19	317	35.2	225
84-5-8M, CL	0.53	0.69	731	39.5	1300

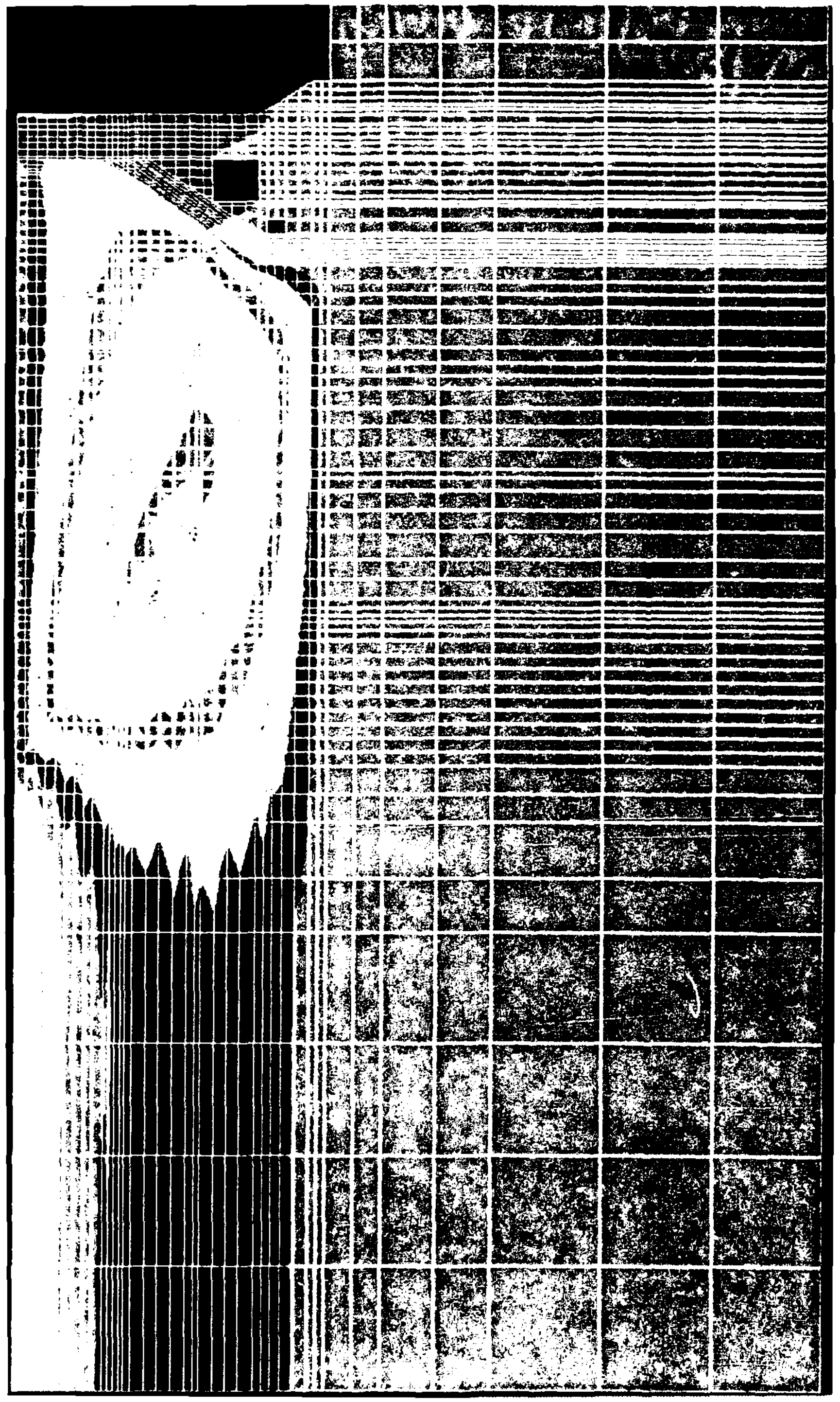
WES FORM NO. 1253
REV OCT 1968

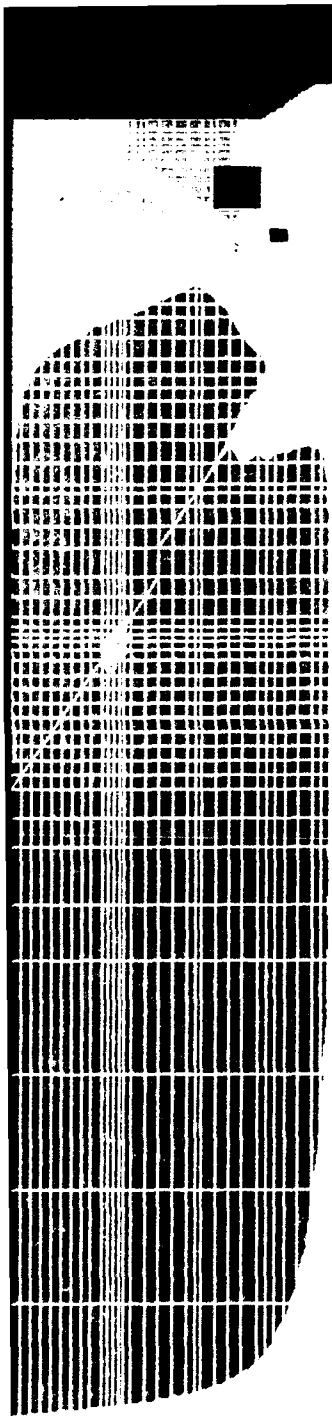
APPENDIX E: RESULTS FOR SOUTH CHAMBER WALL AT EISENHOWER LOCK

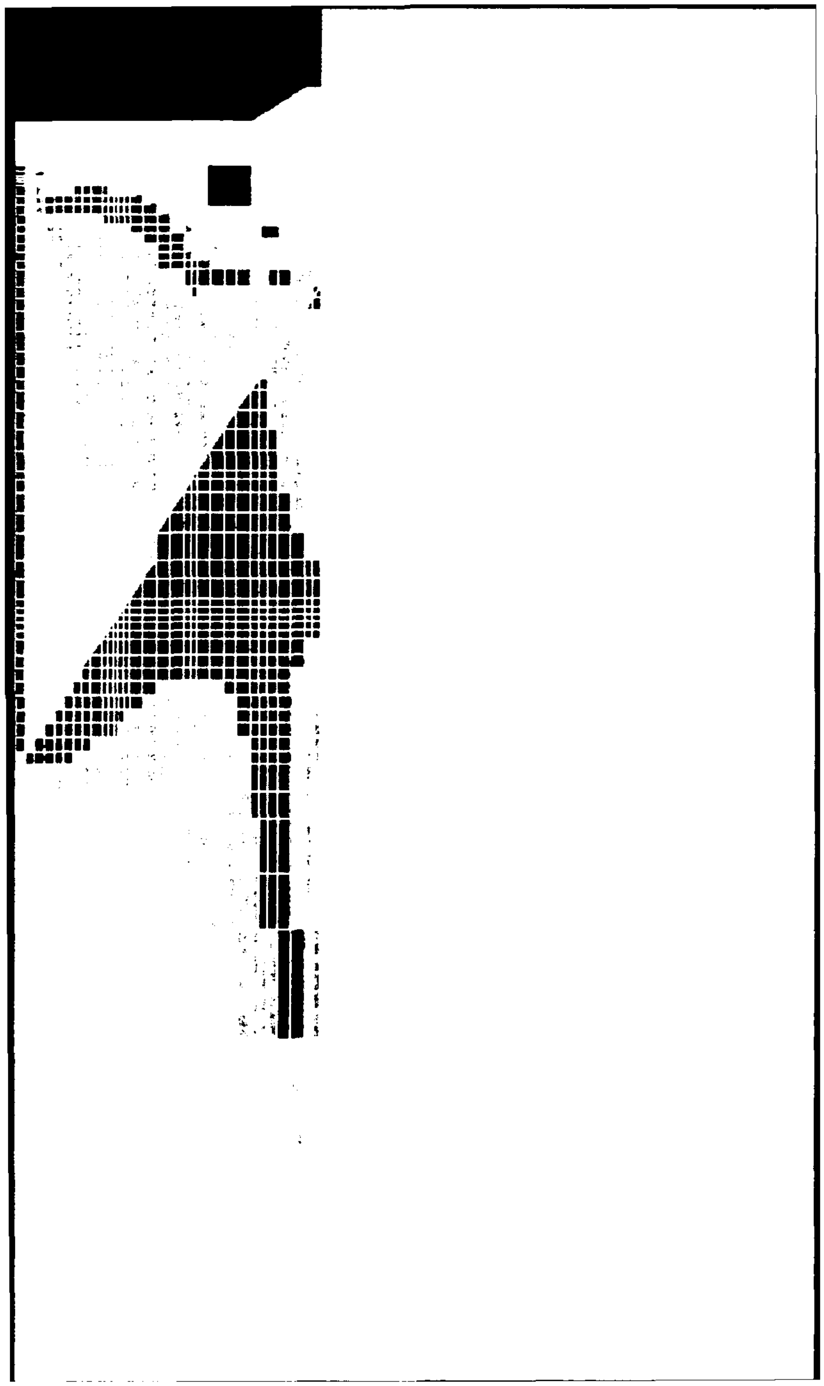


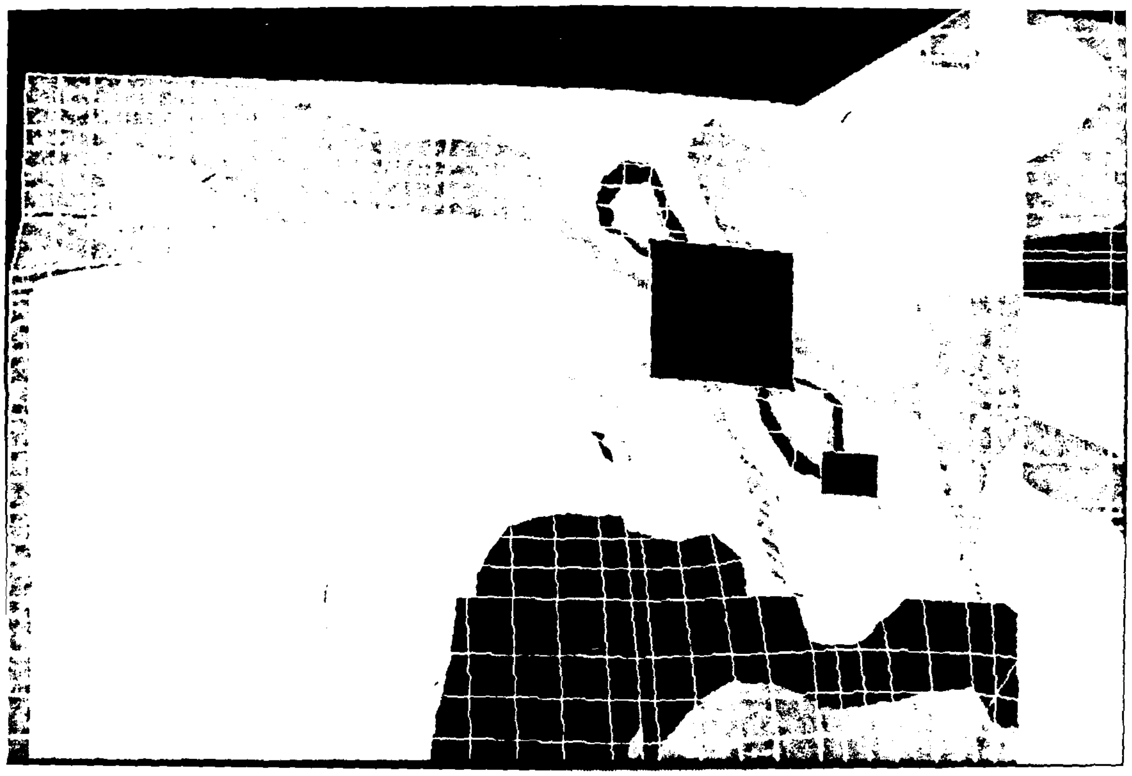


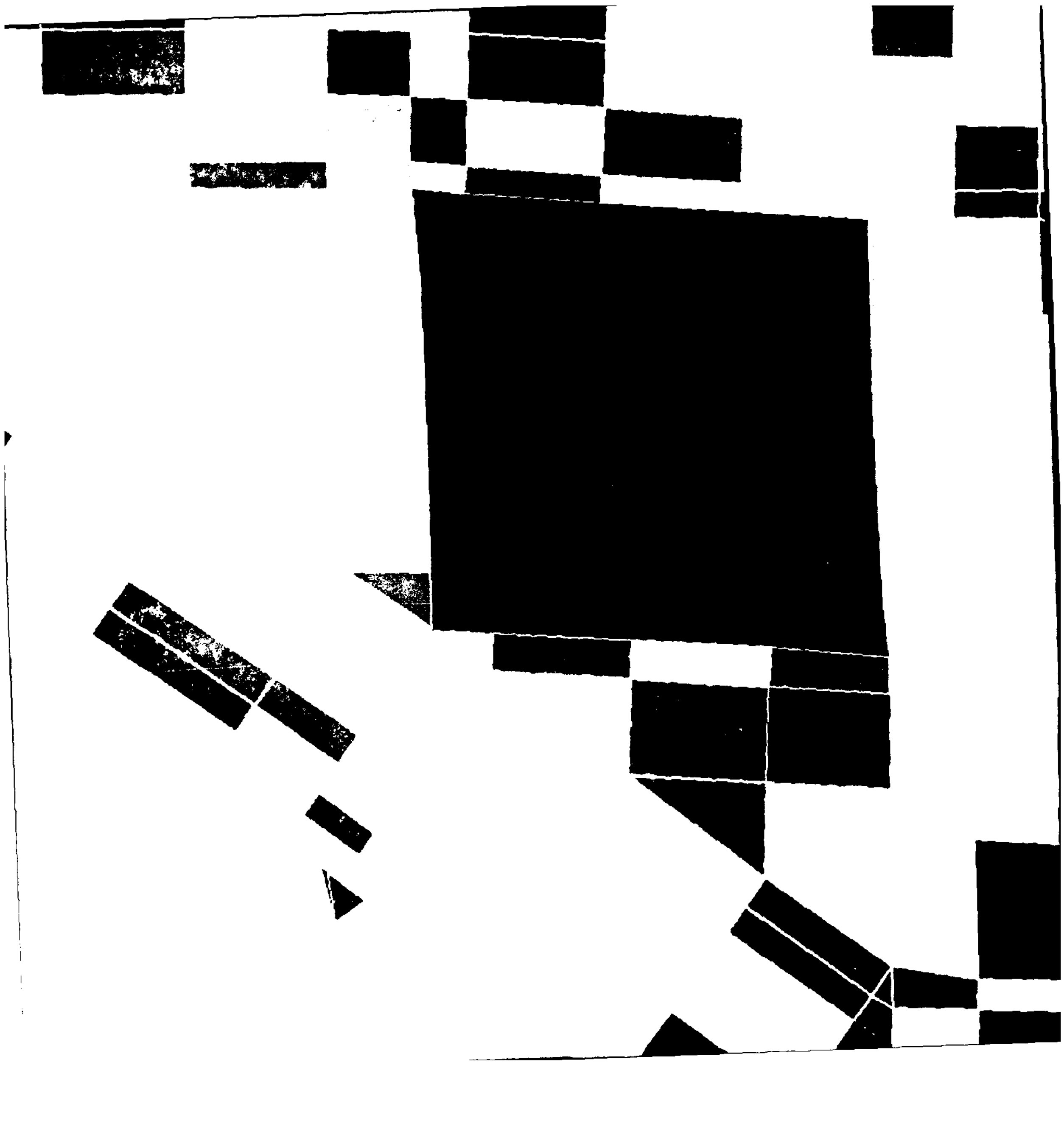


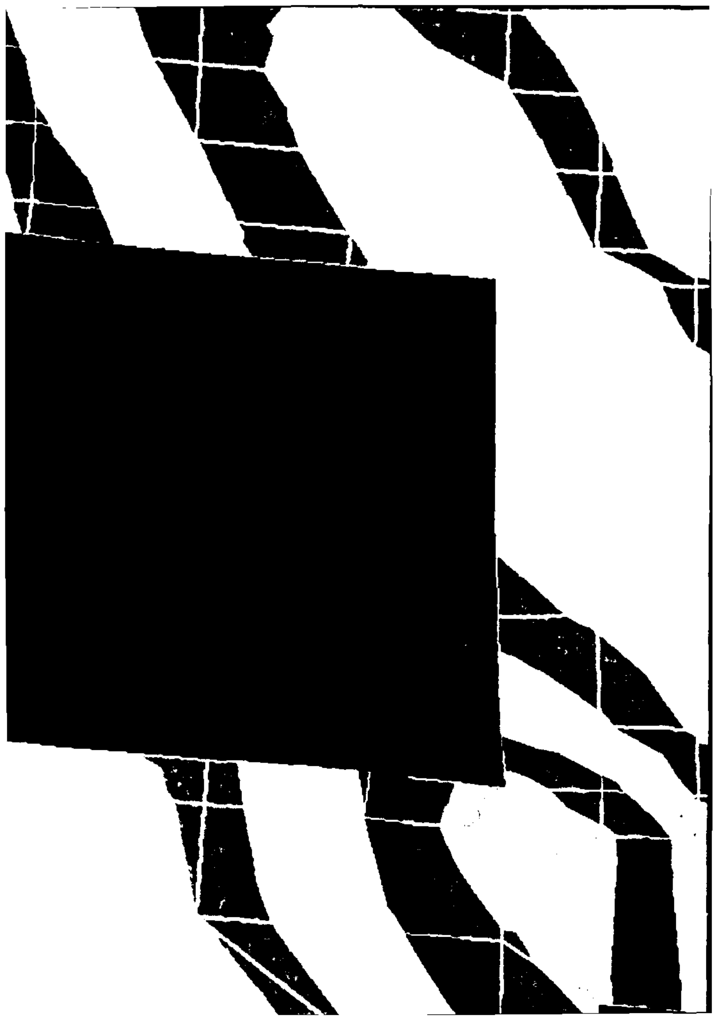


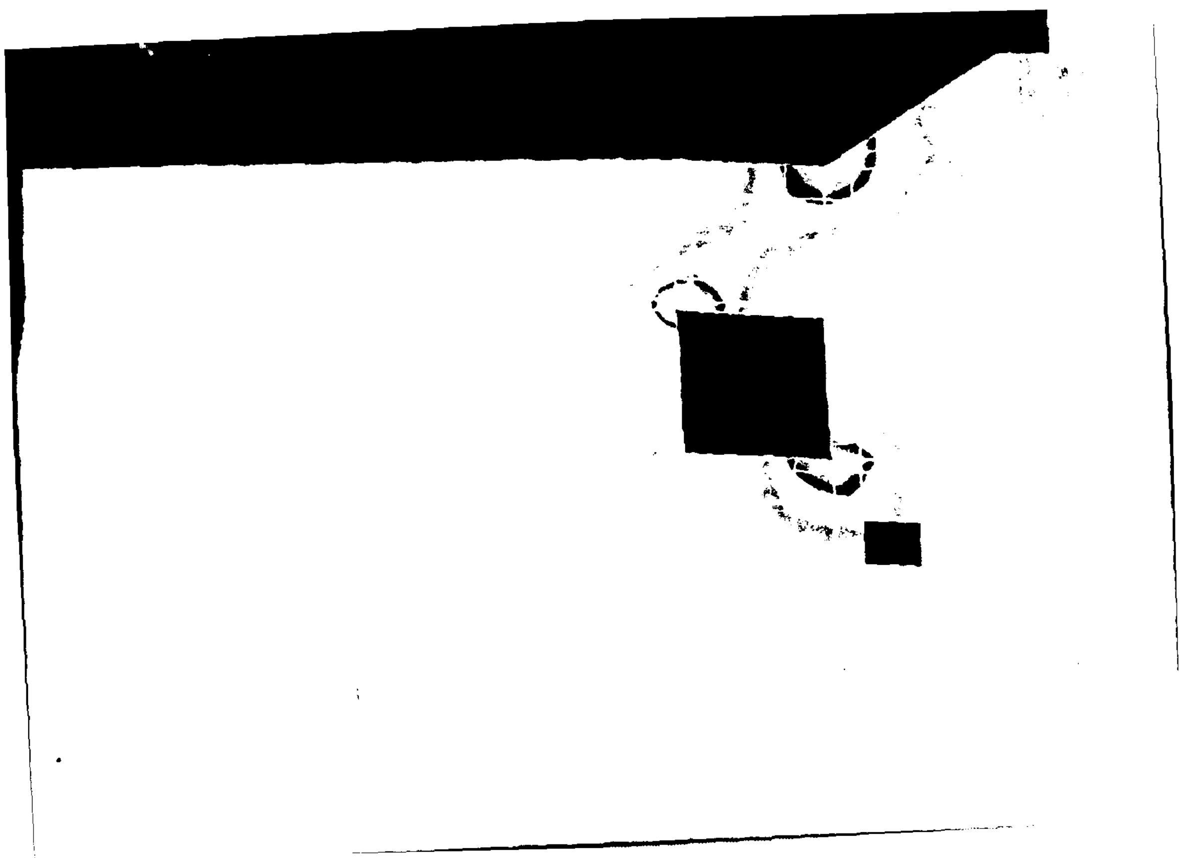


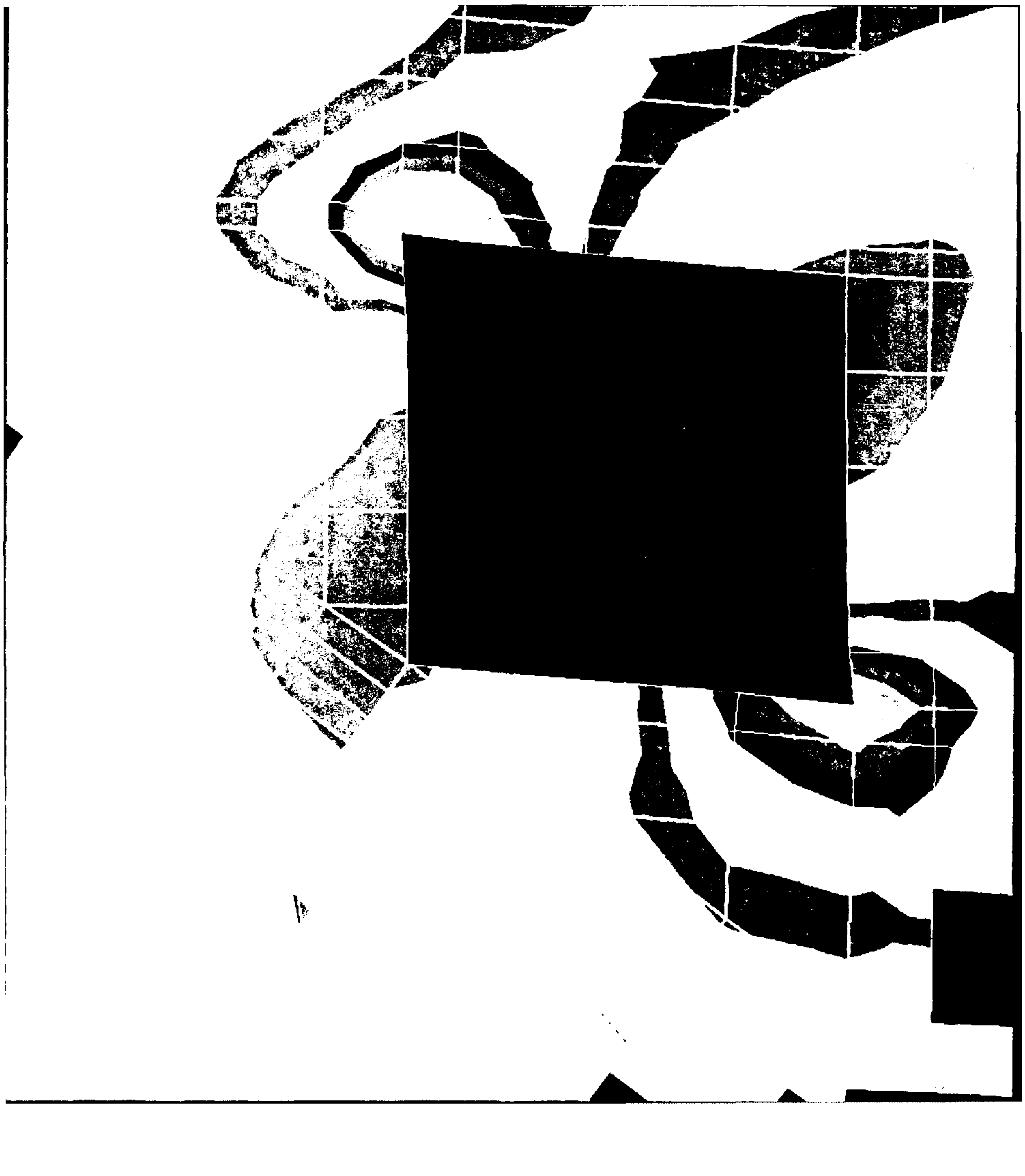


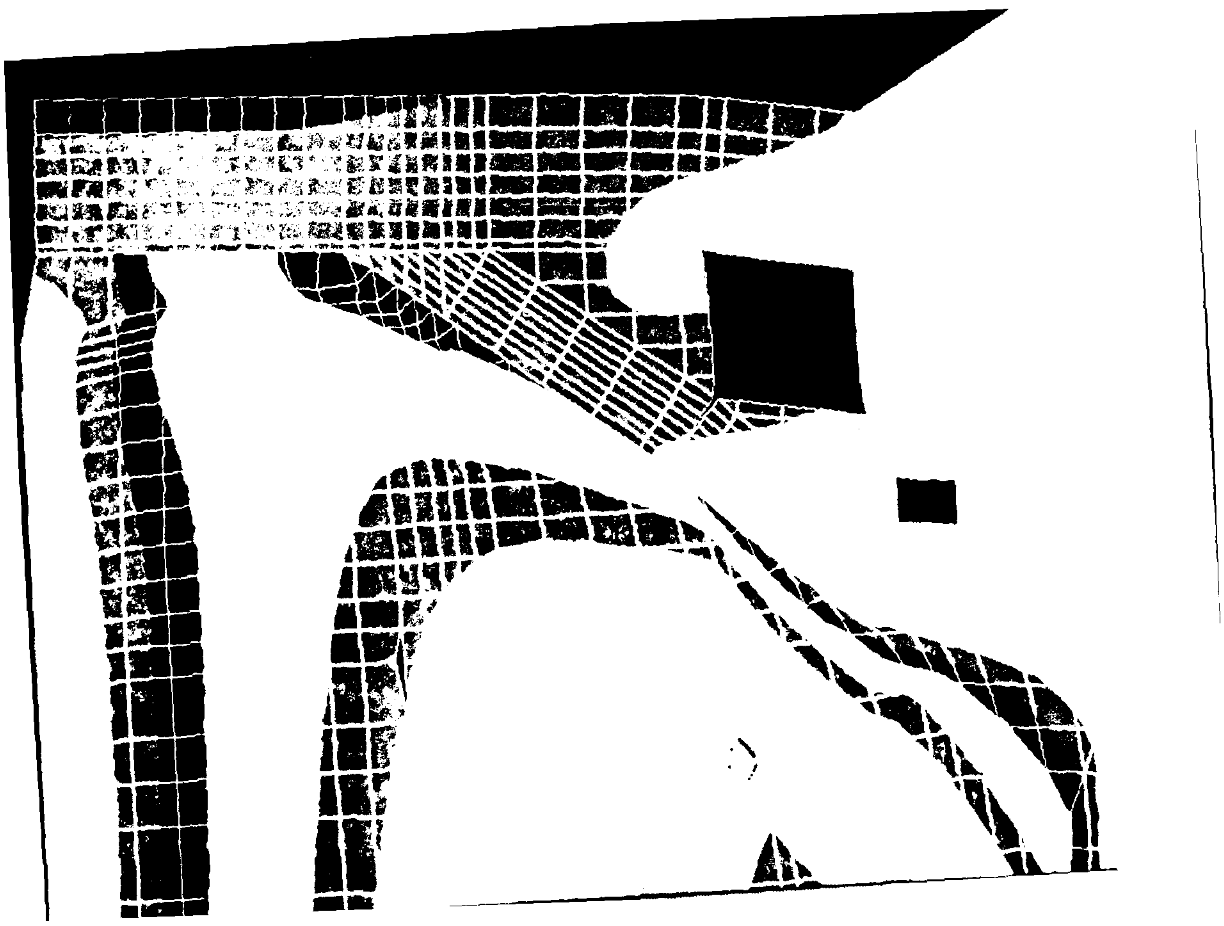


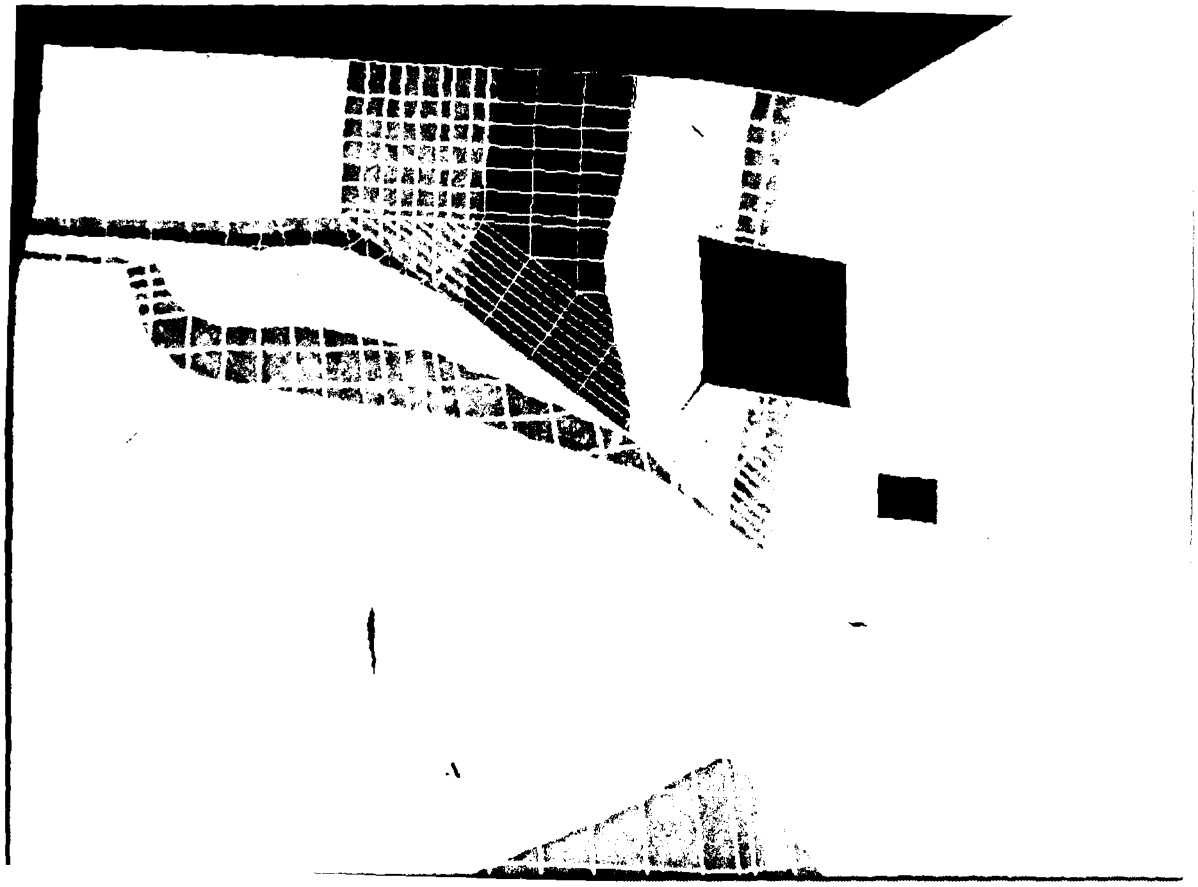


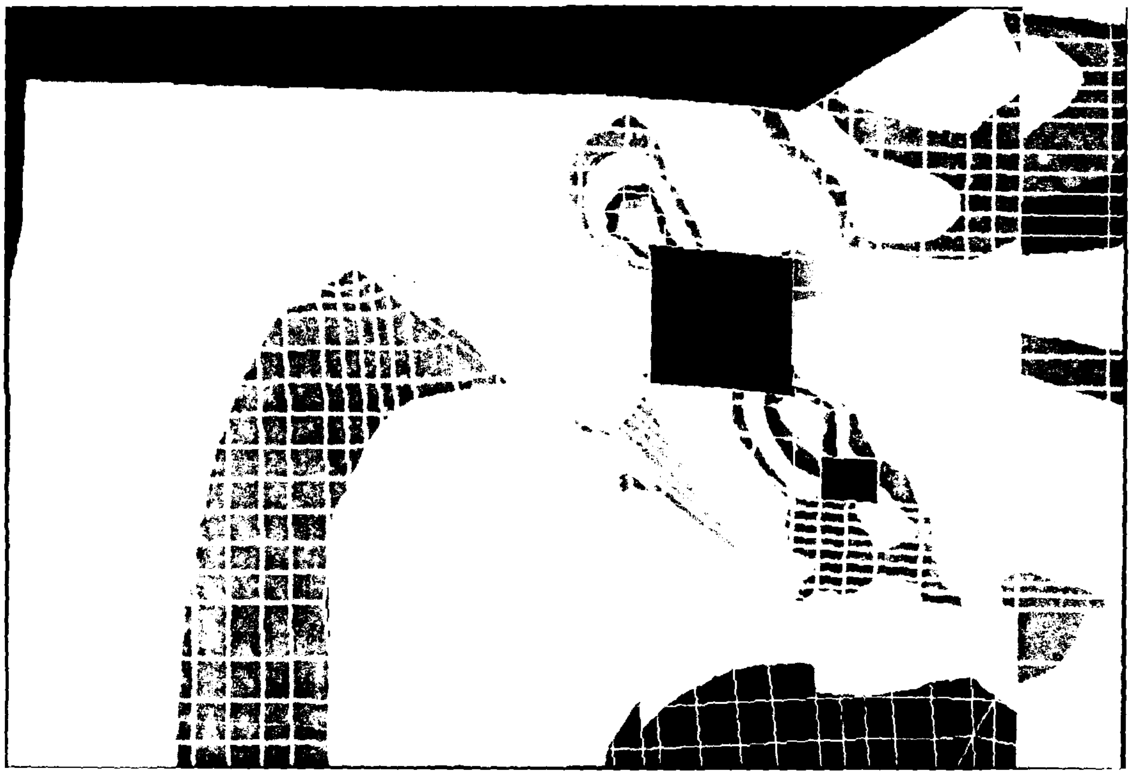


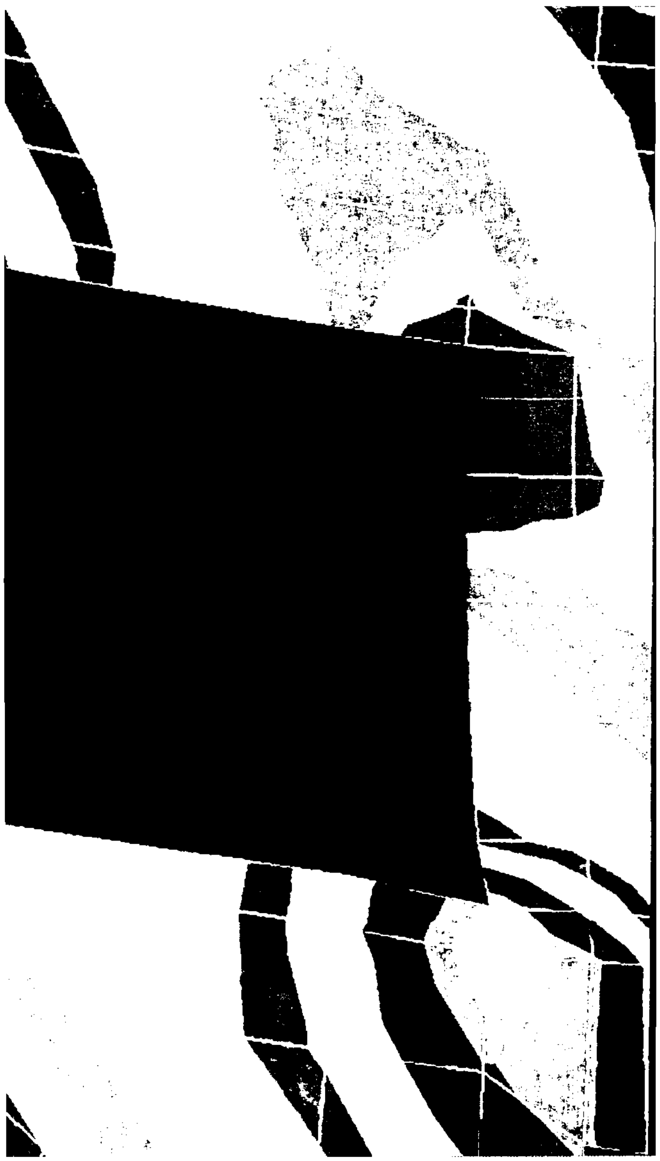


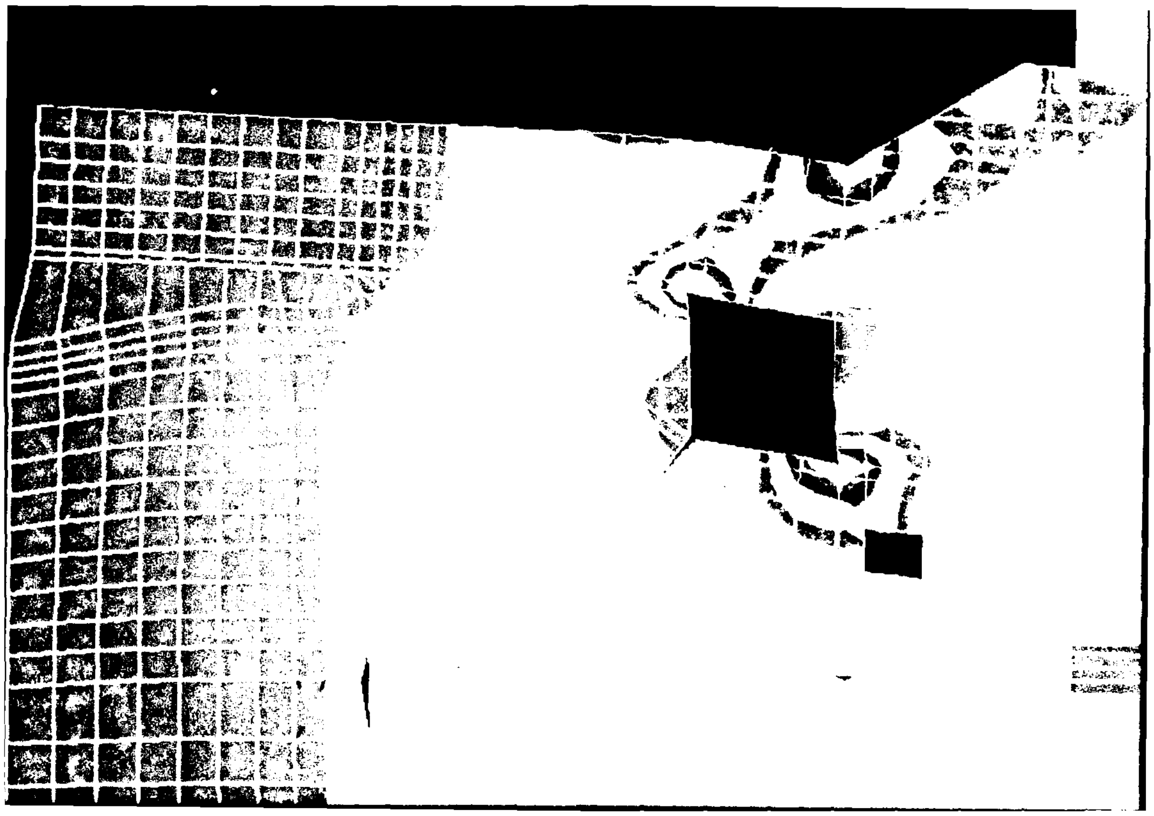


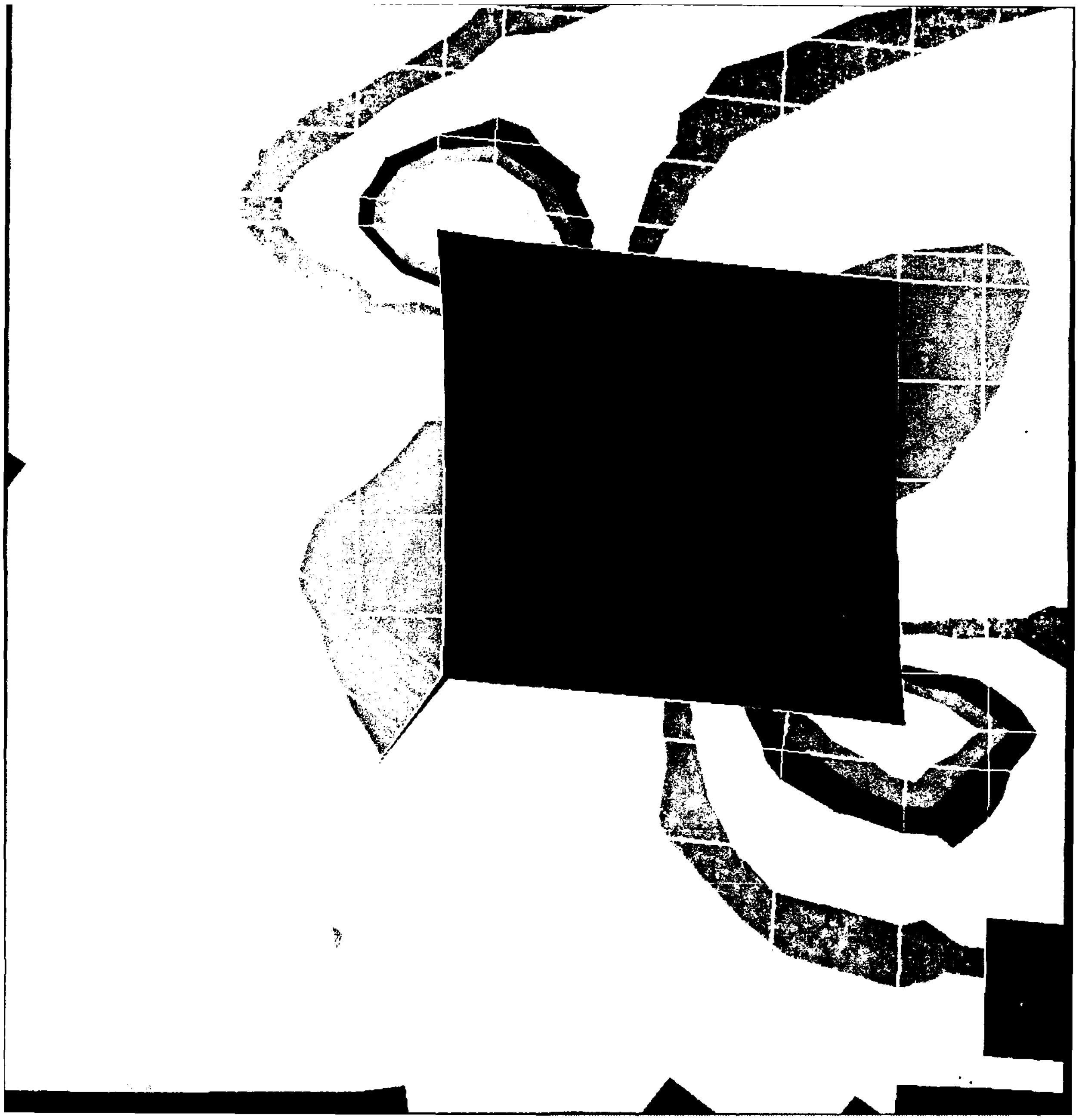


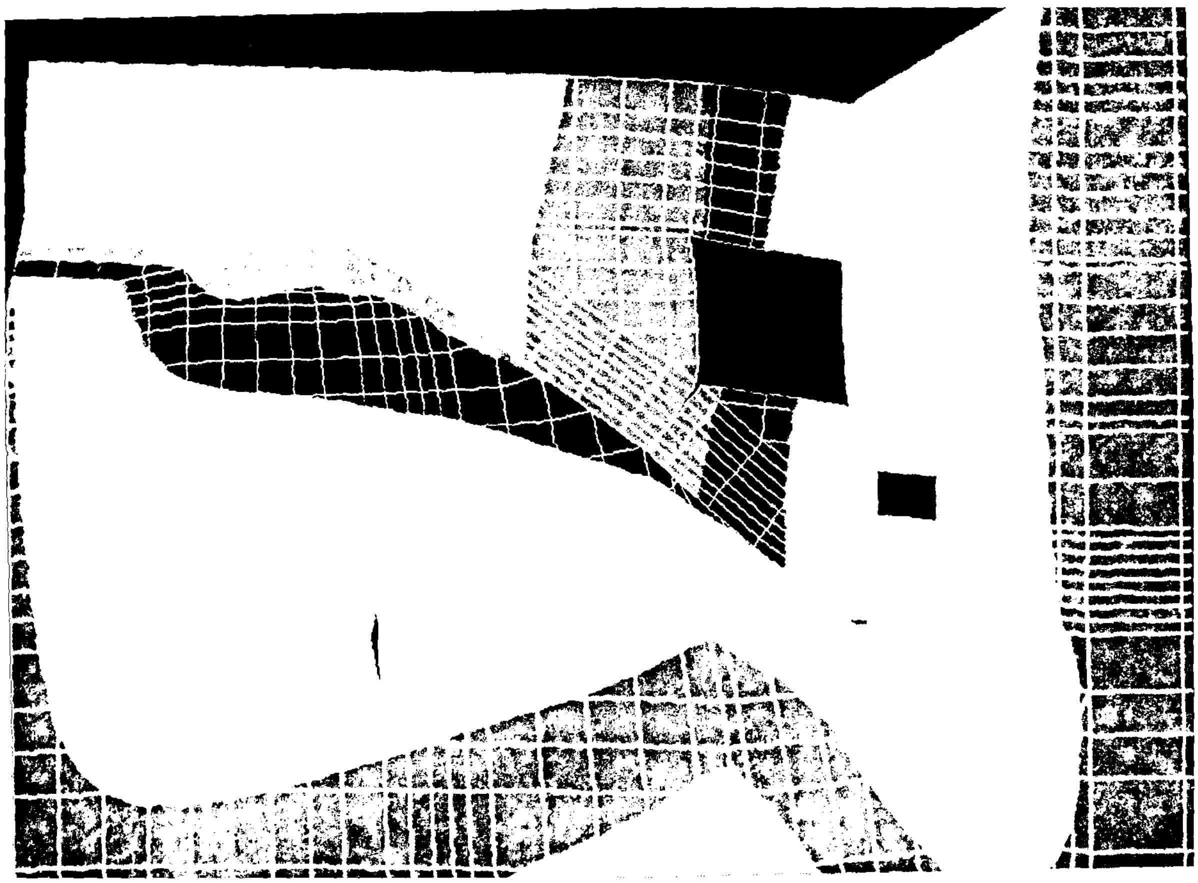


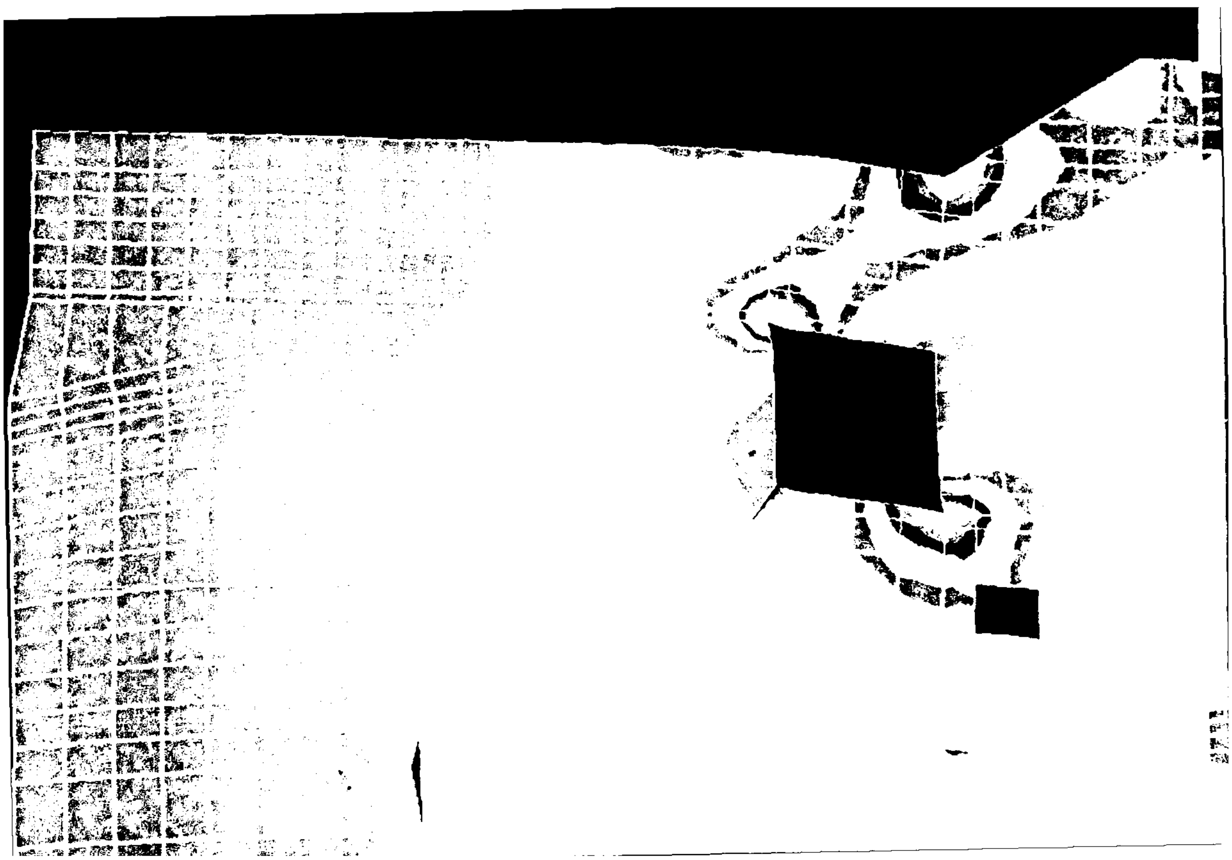












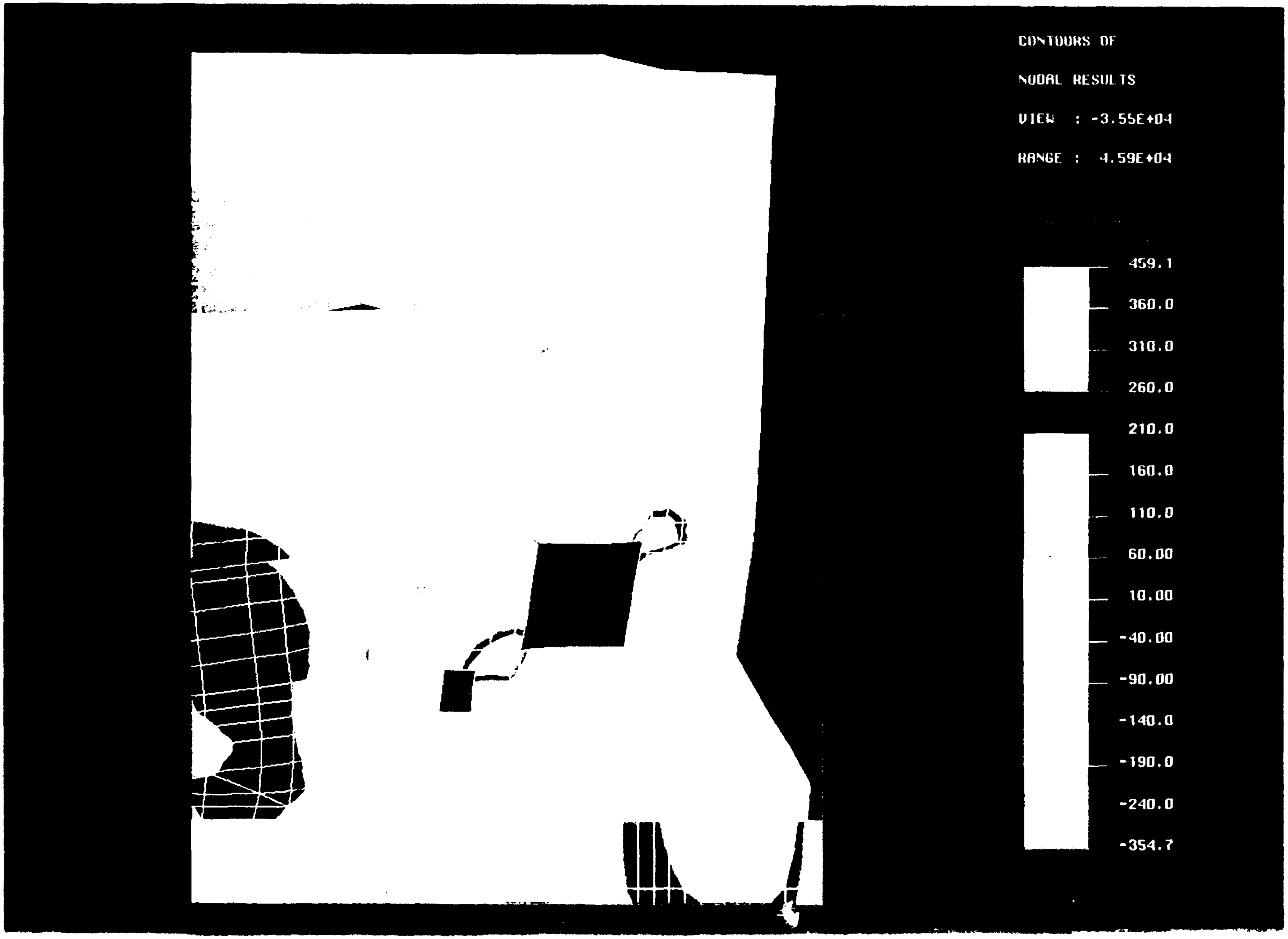
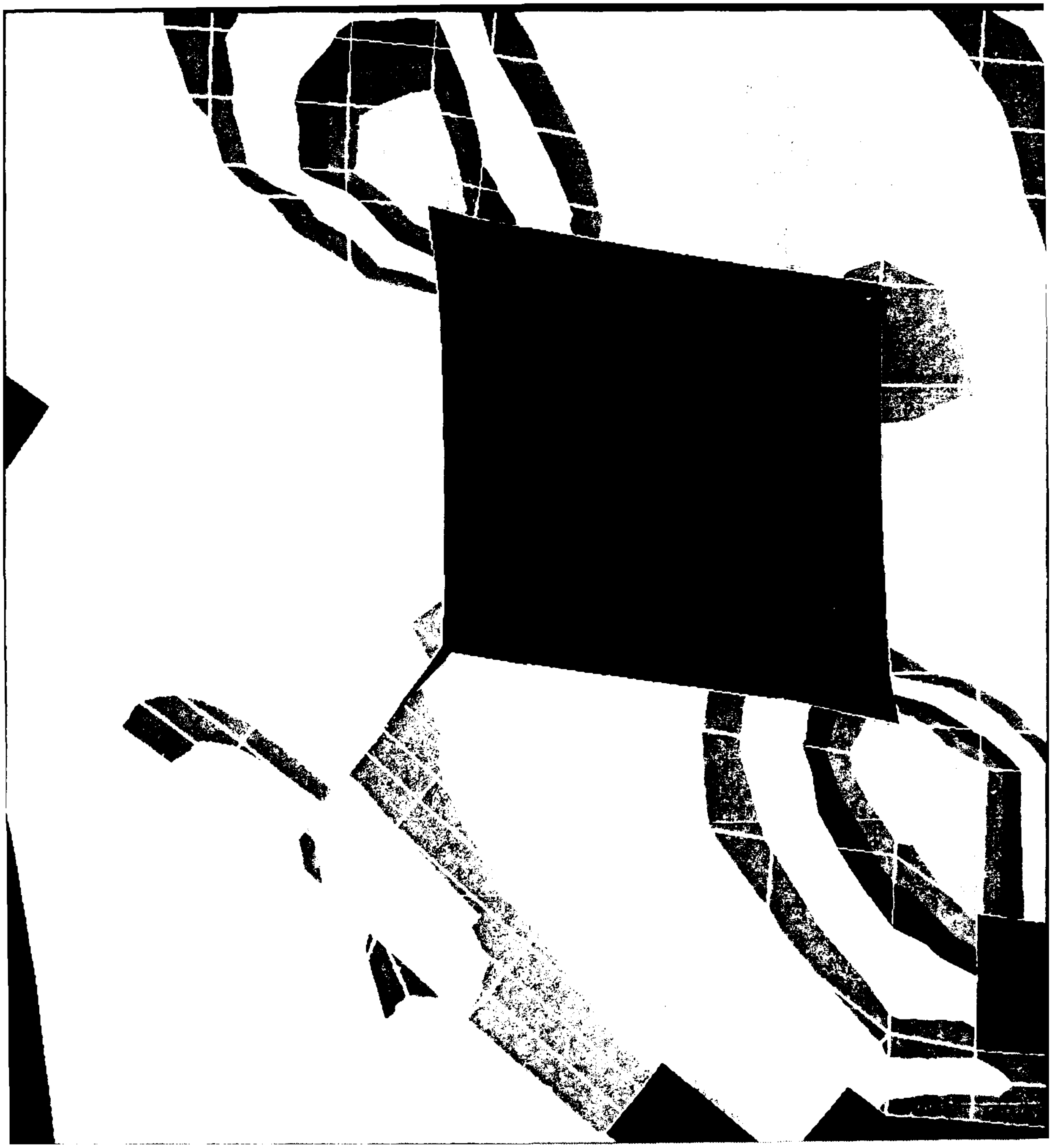
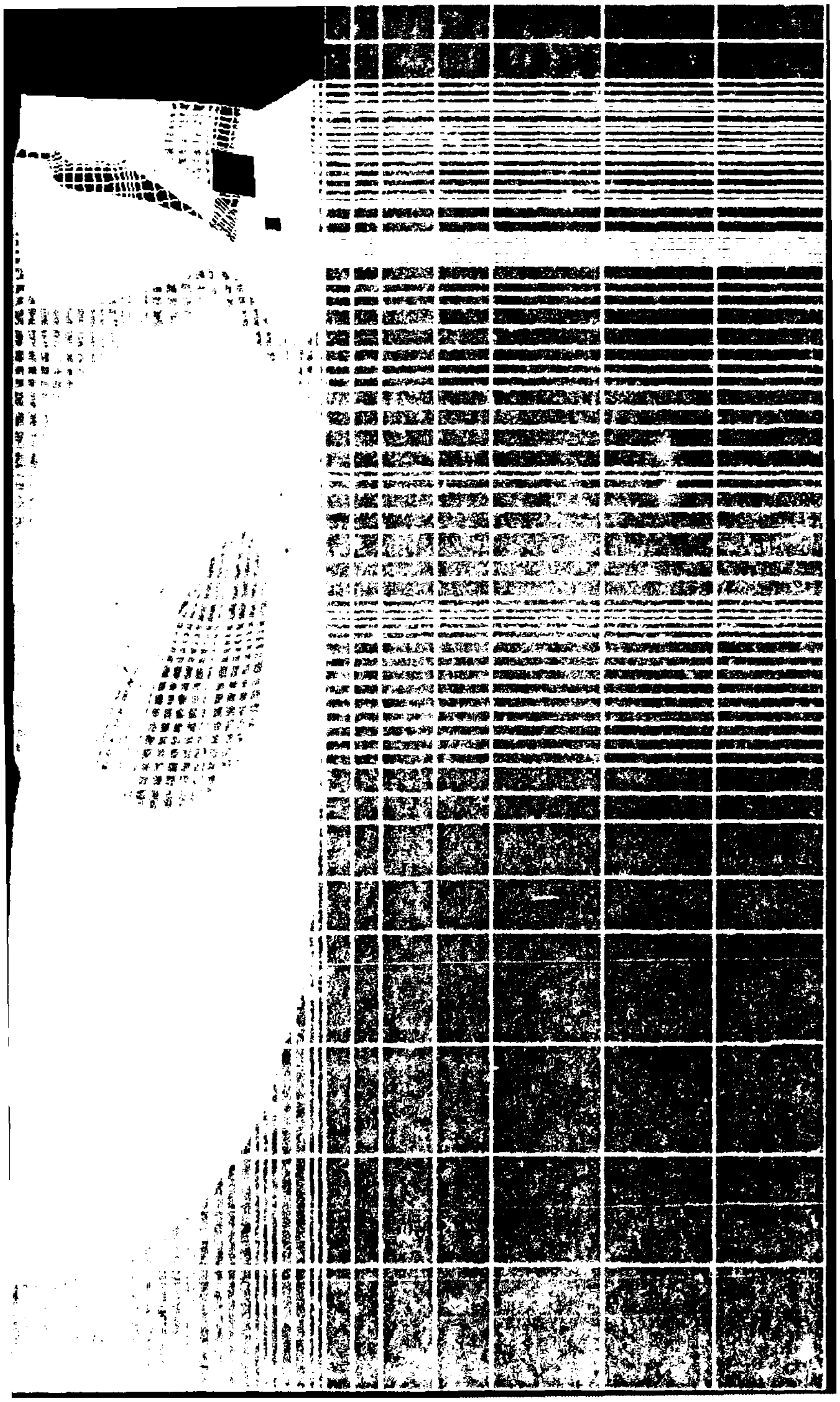
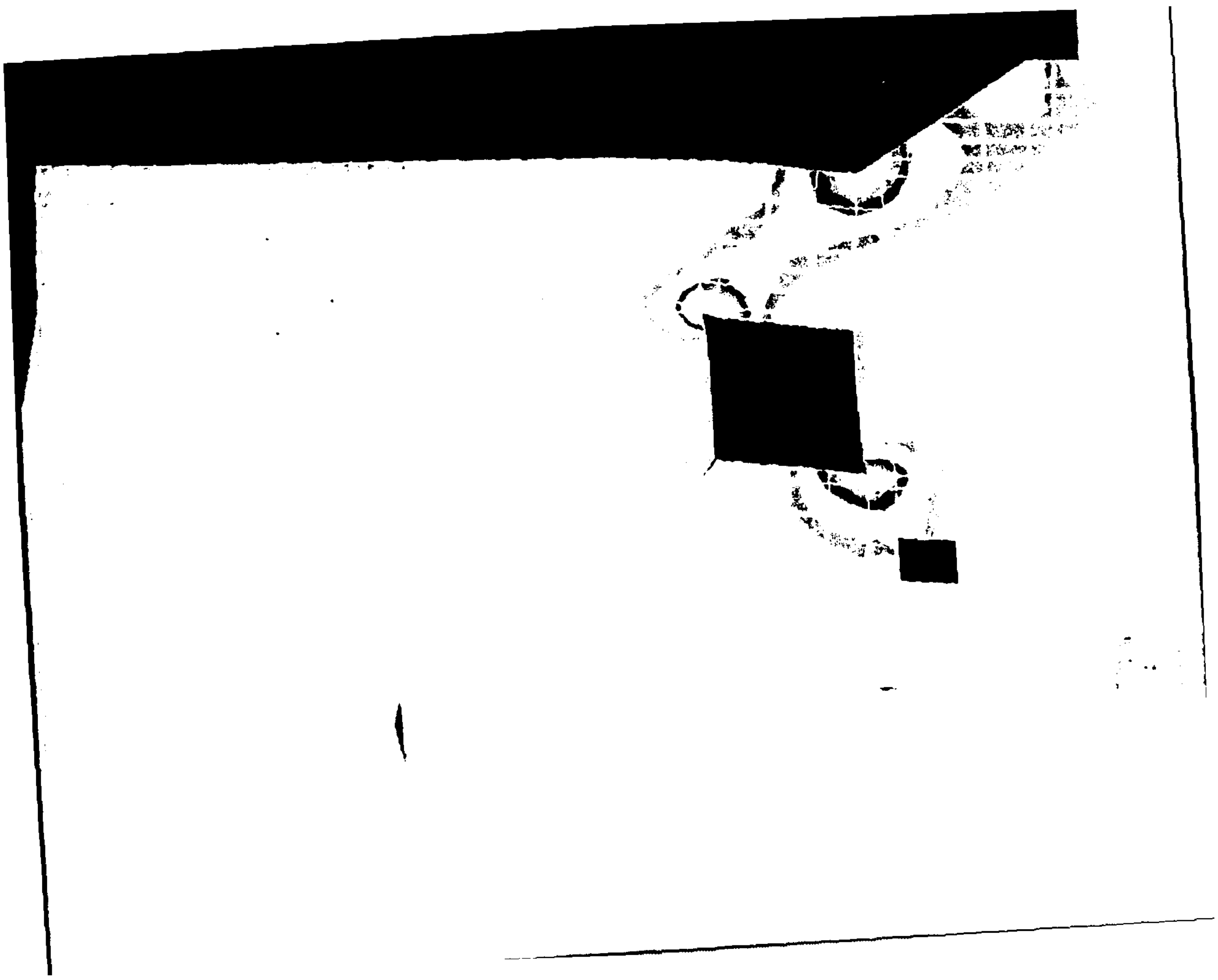
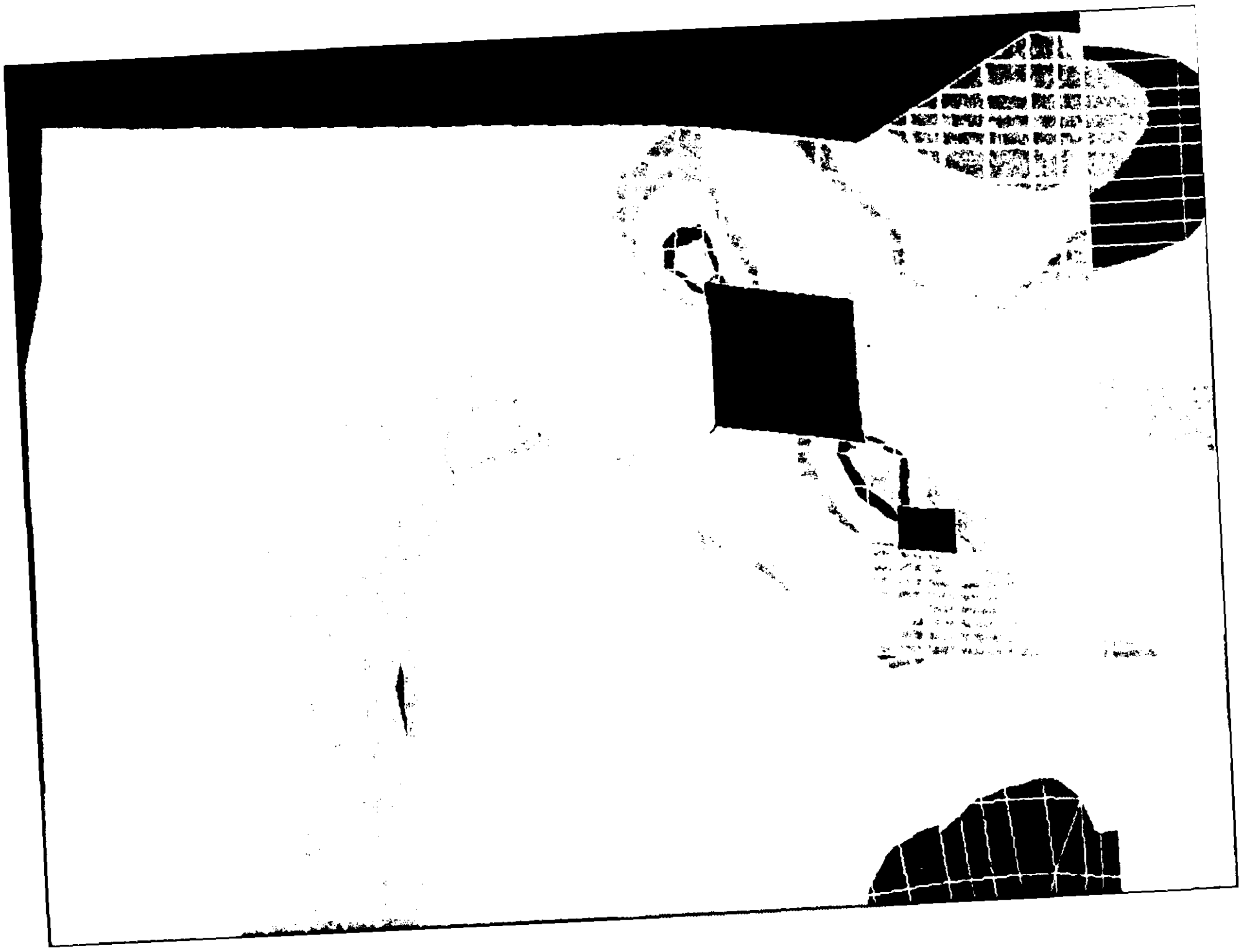


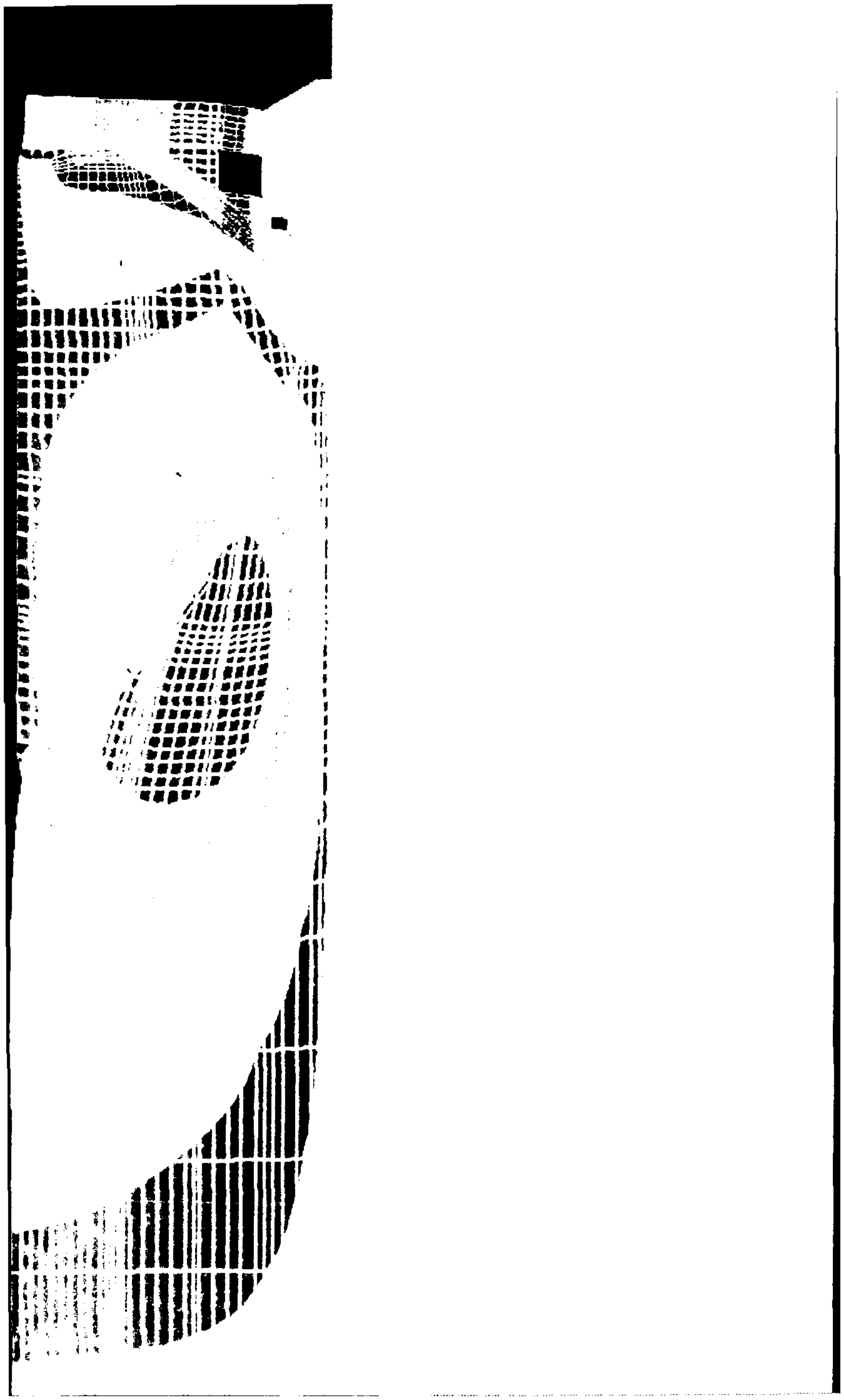
Figure E20. Distribution of minor principal stresses and the deformed shape of the wall and surrounding backfill after placement of the anchors

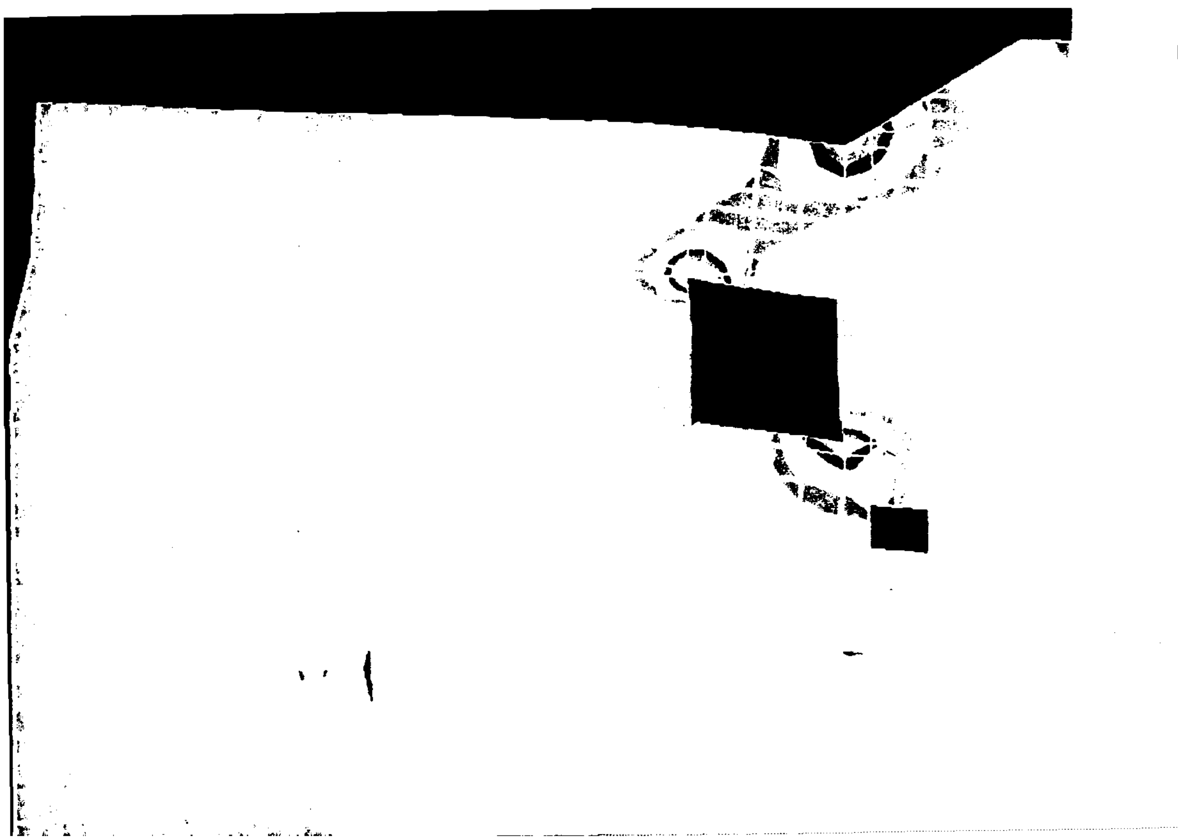


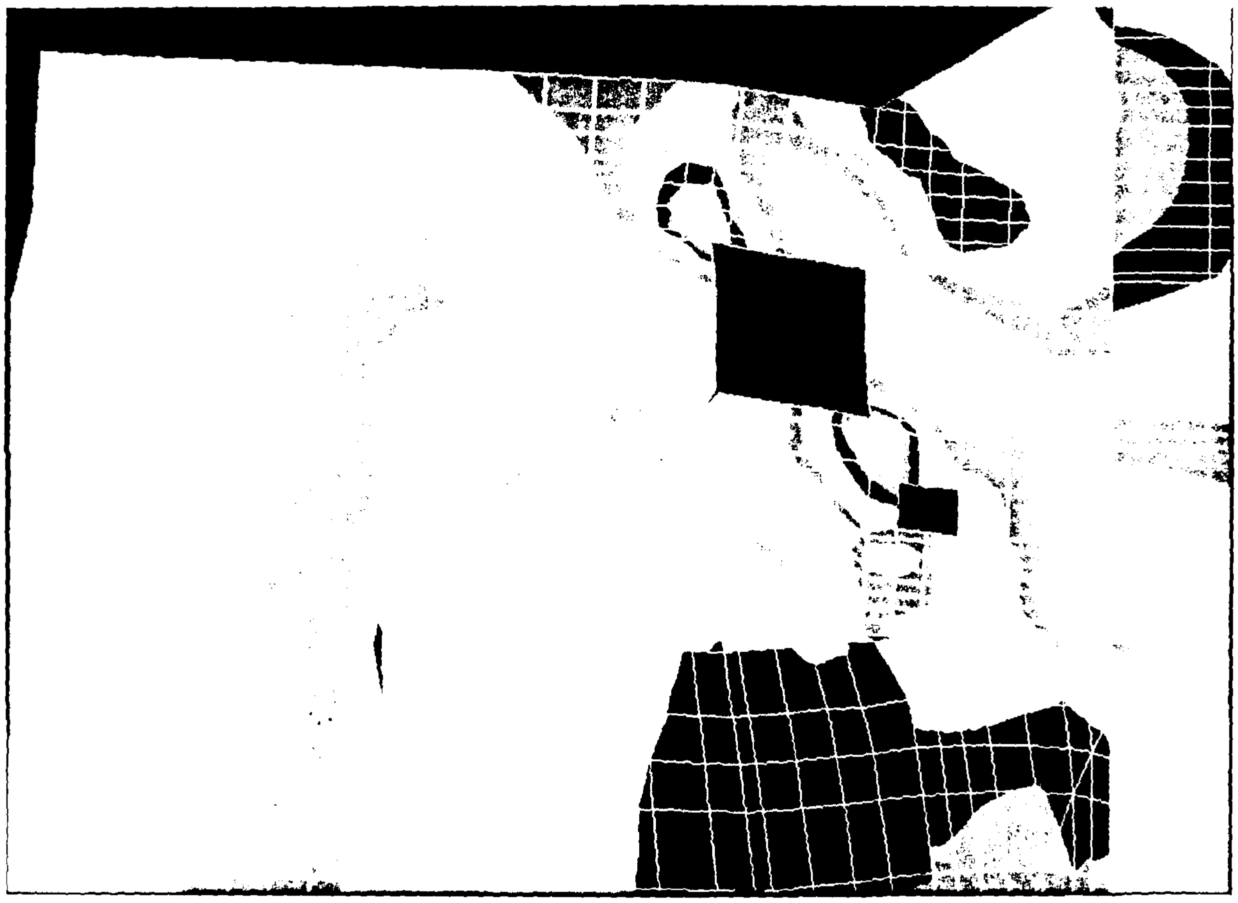




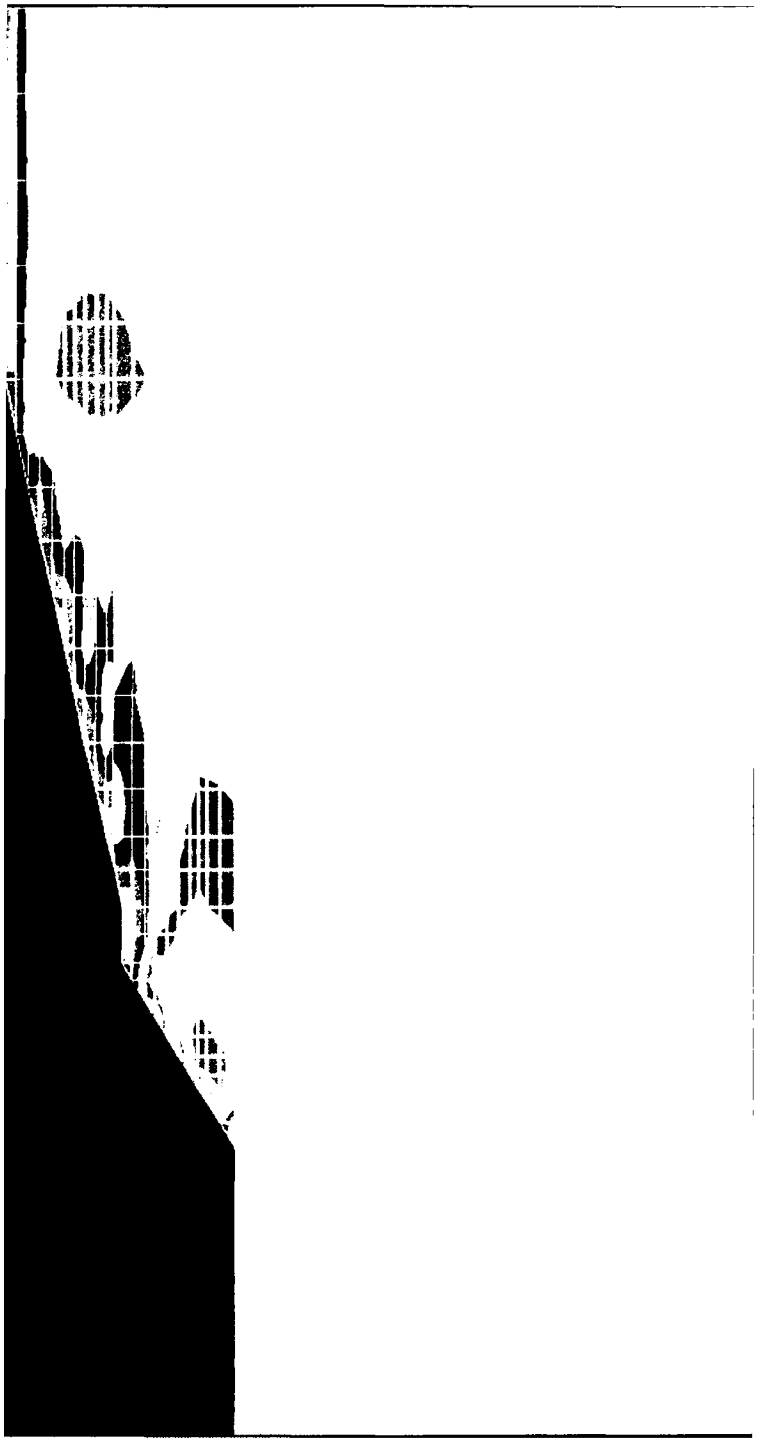








APPENDIX F: RESULTS FOR NORTH CHAMBER WALL AT SNELL LOCK



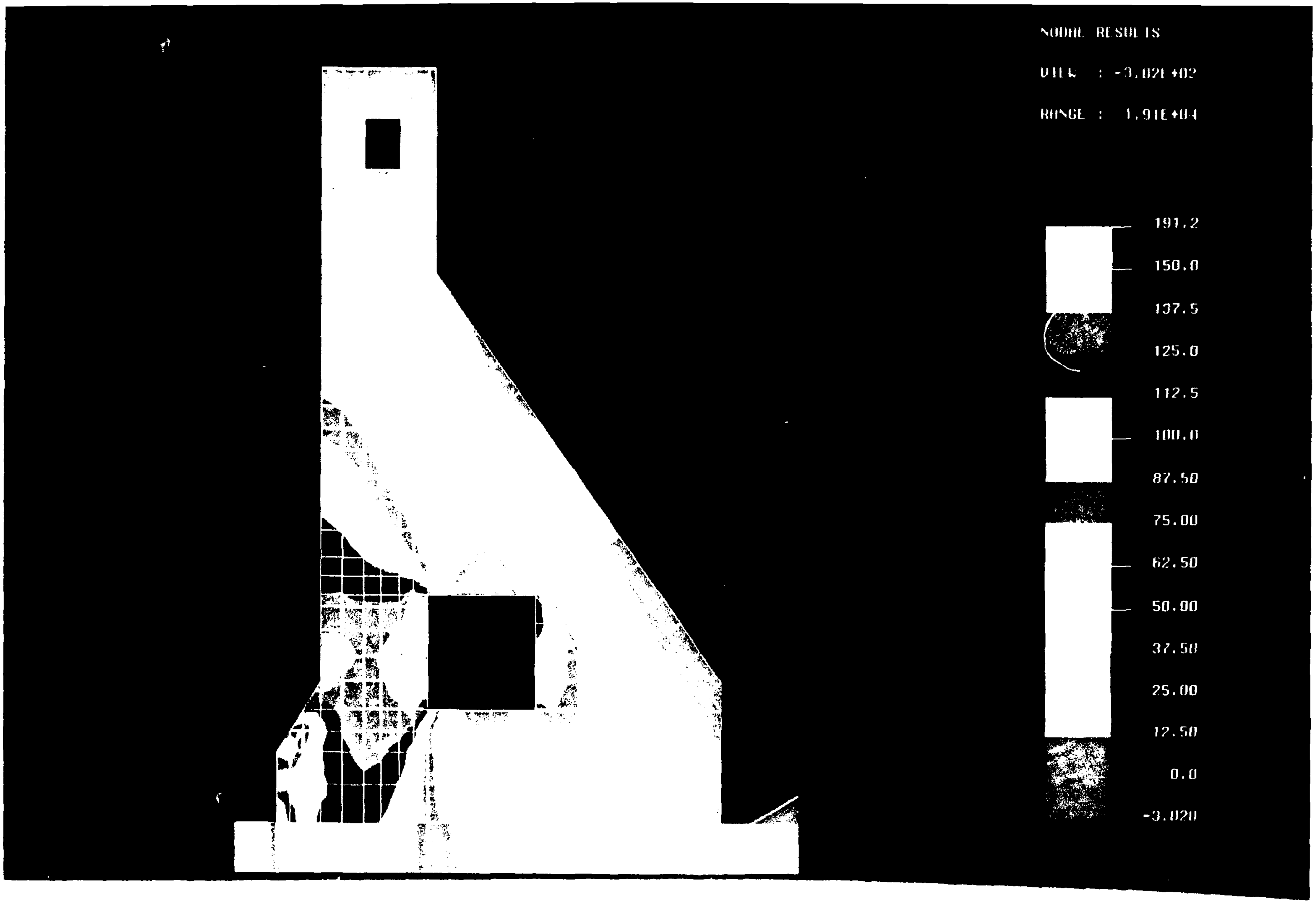
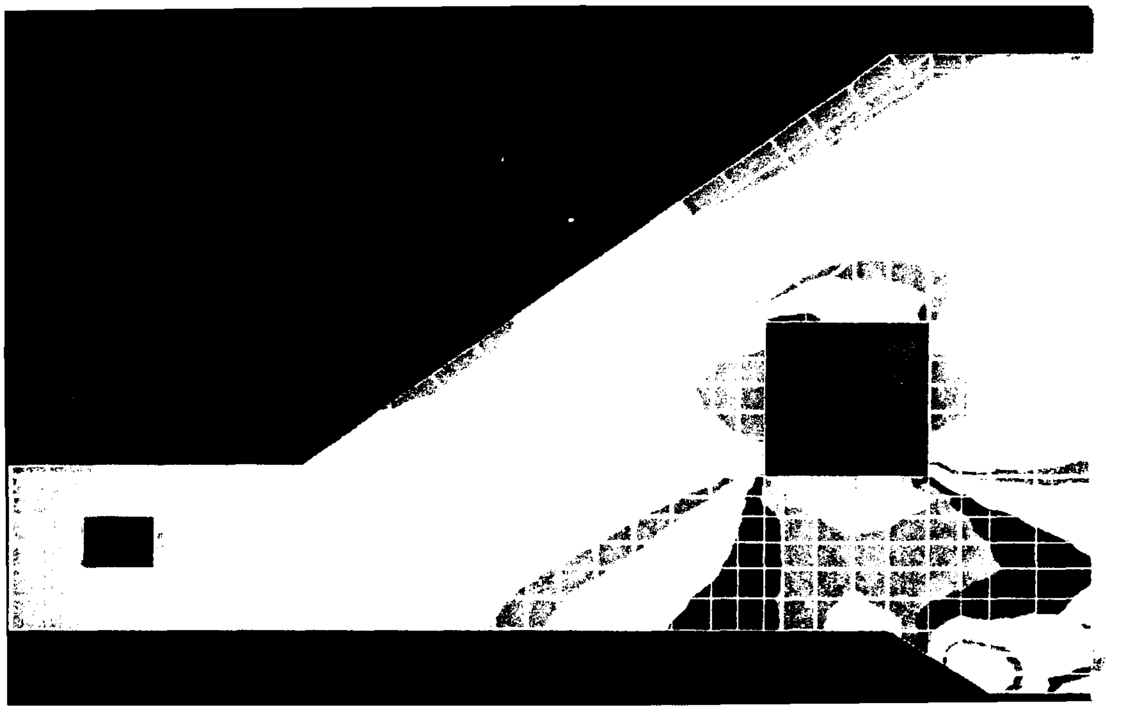
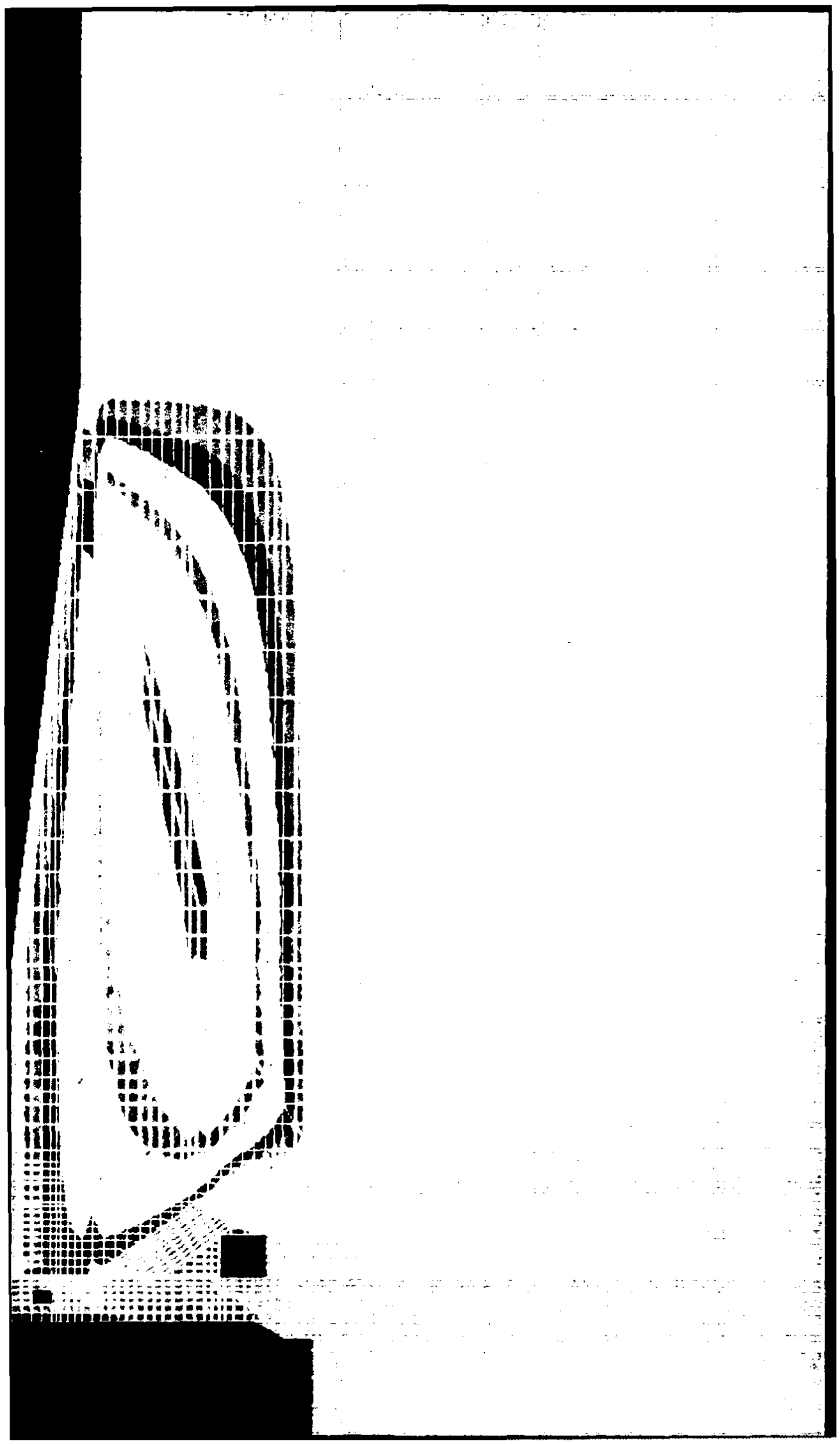
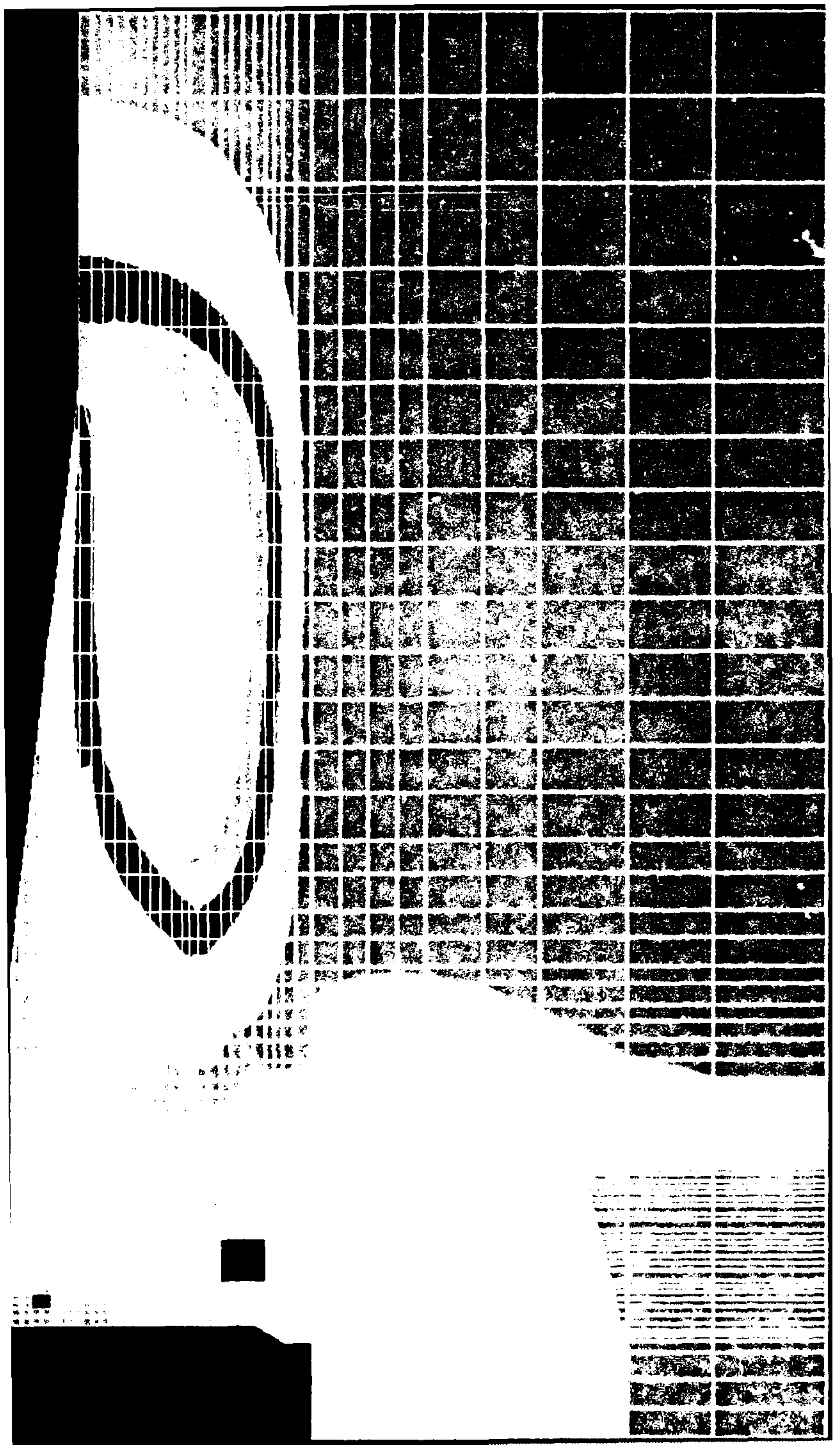
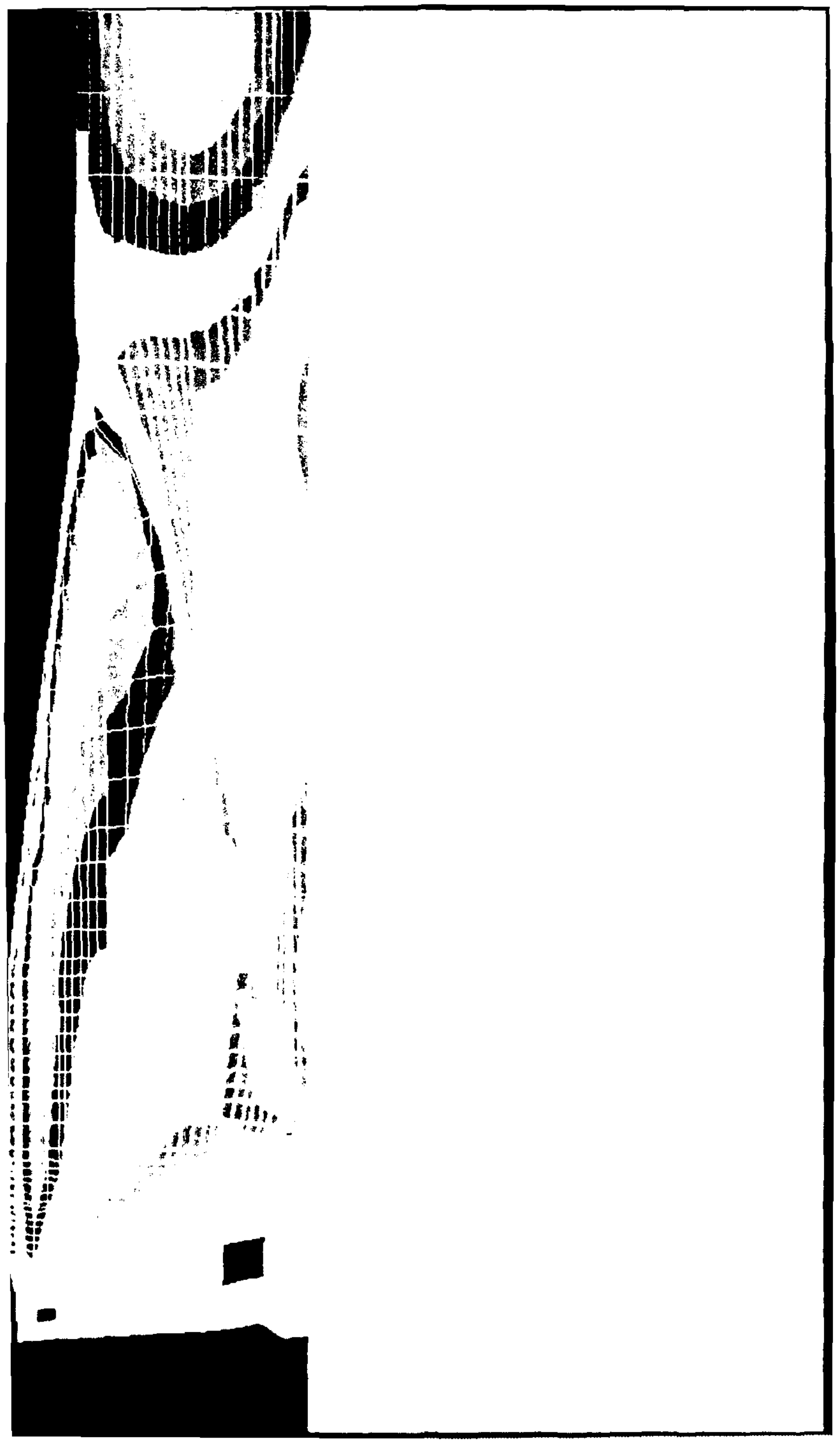


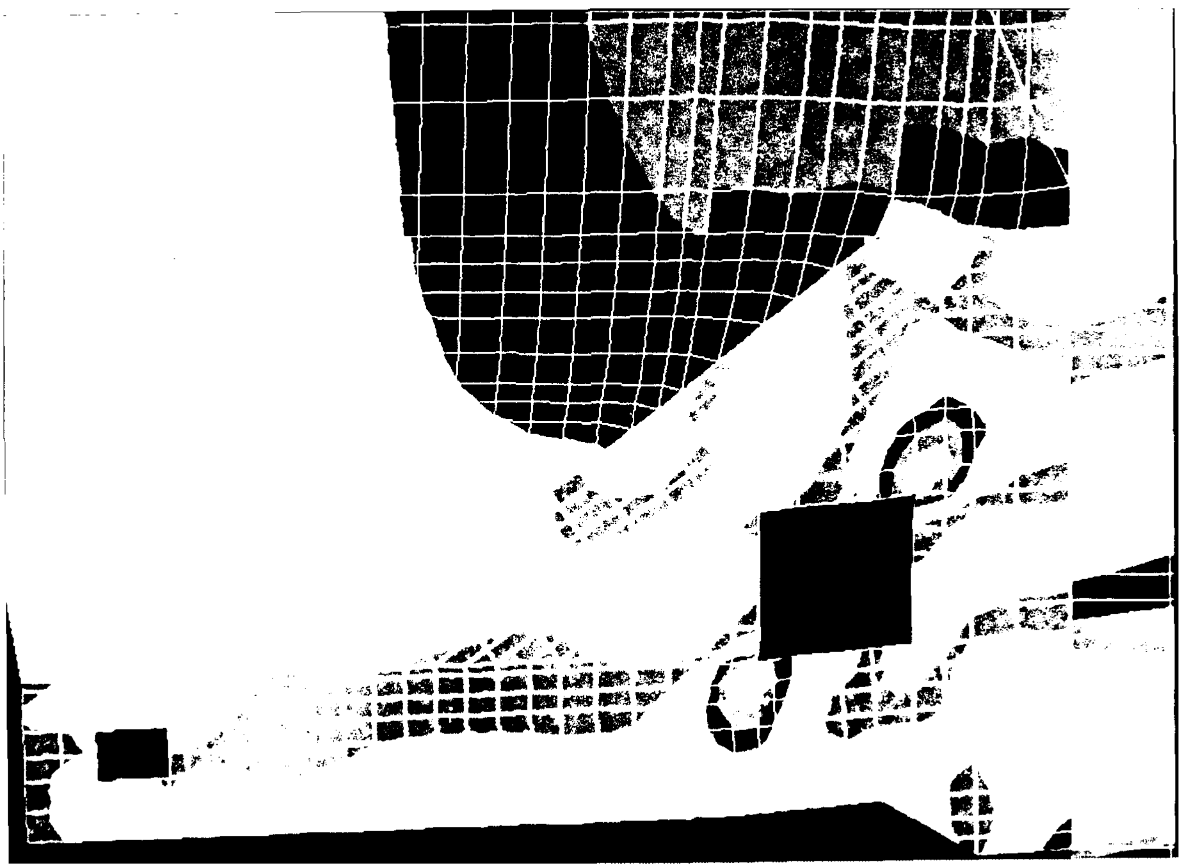
Figure F2. Distribution of vertical stresses in the lock wall

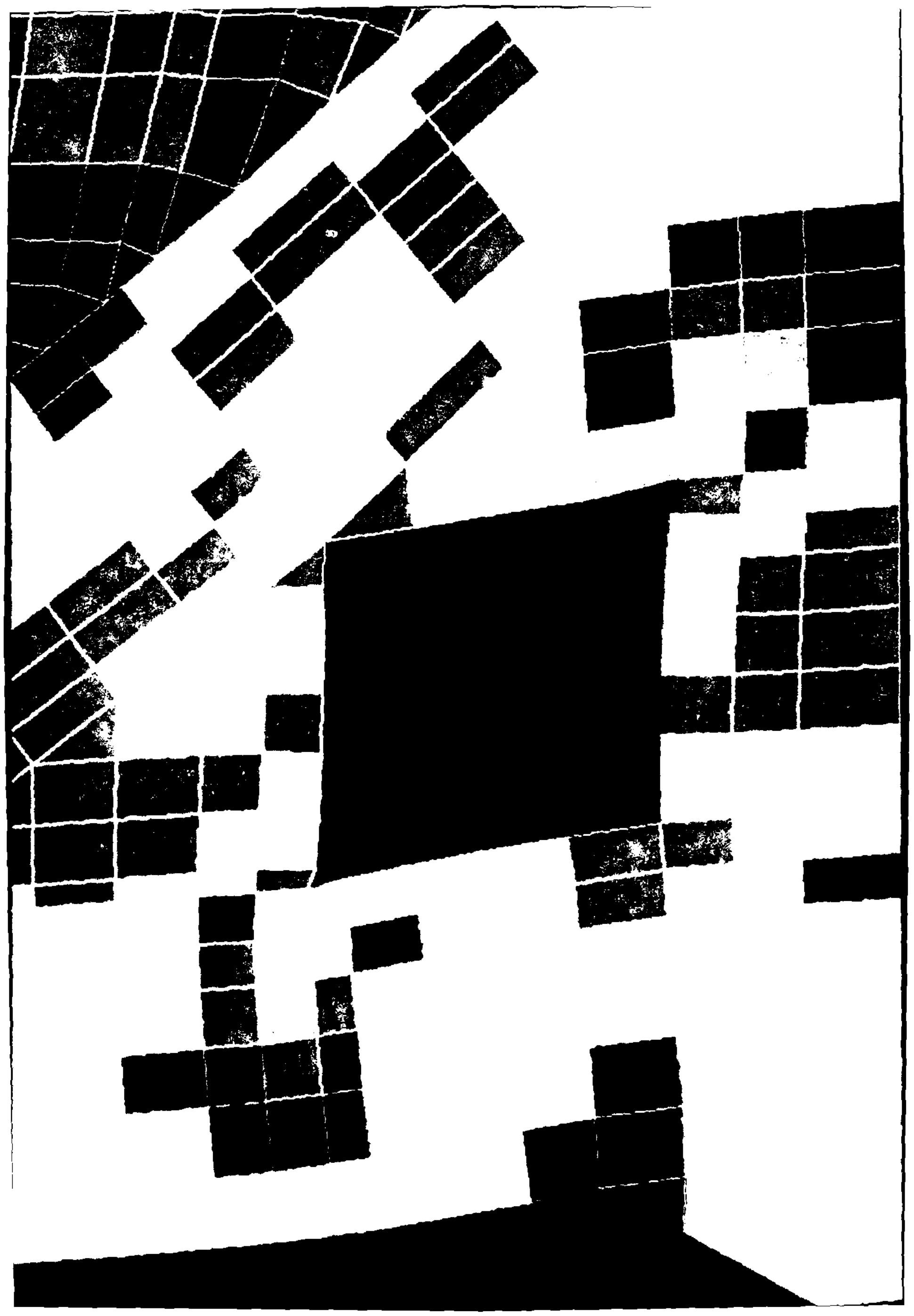


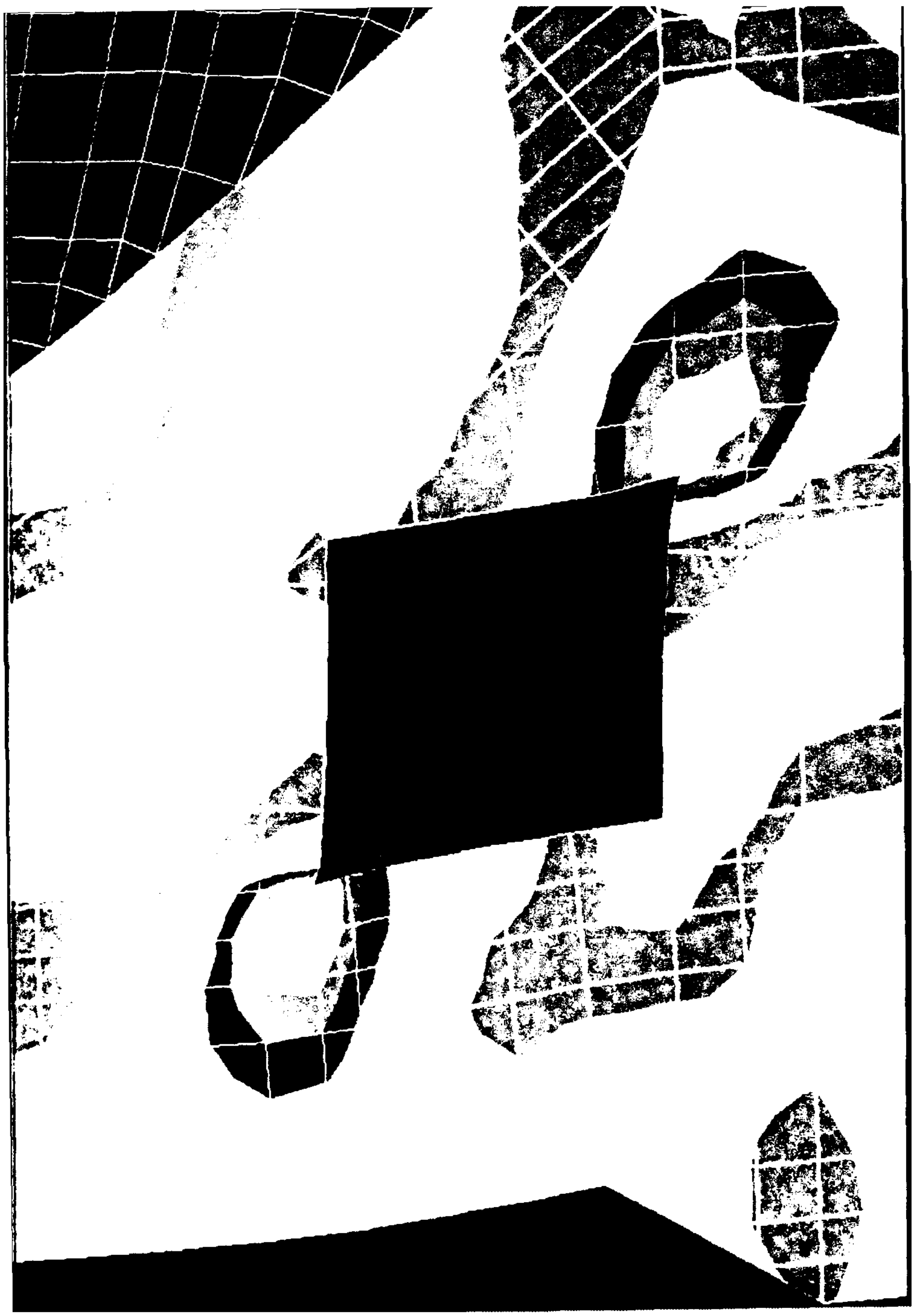


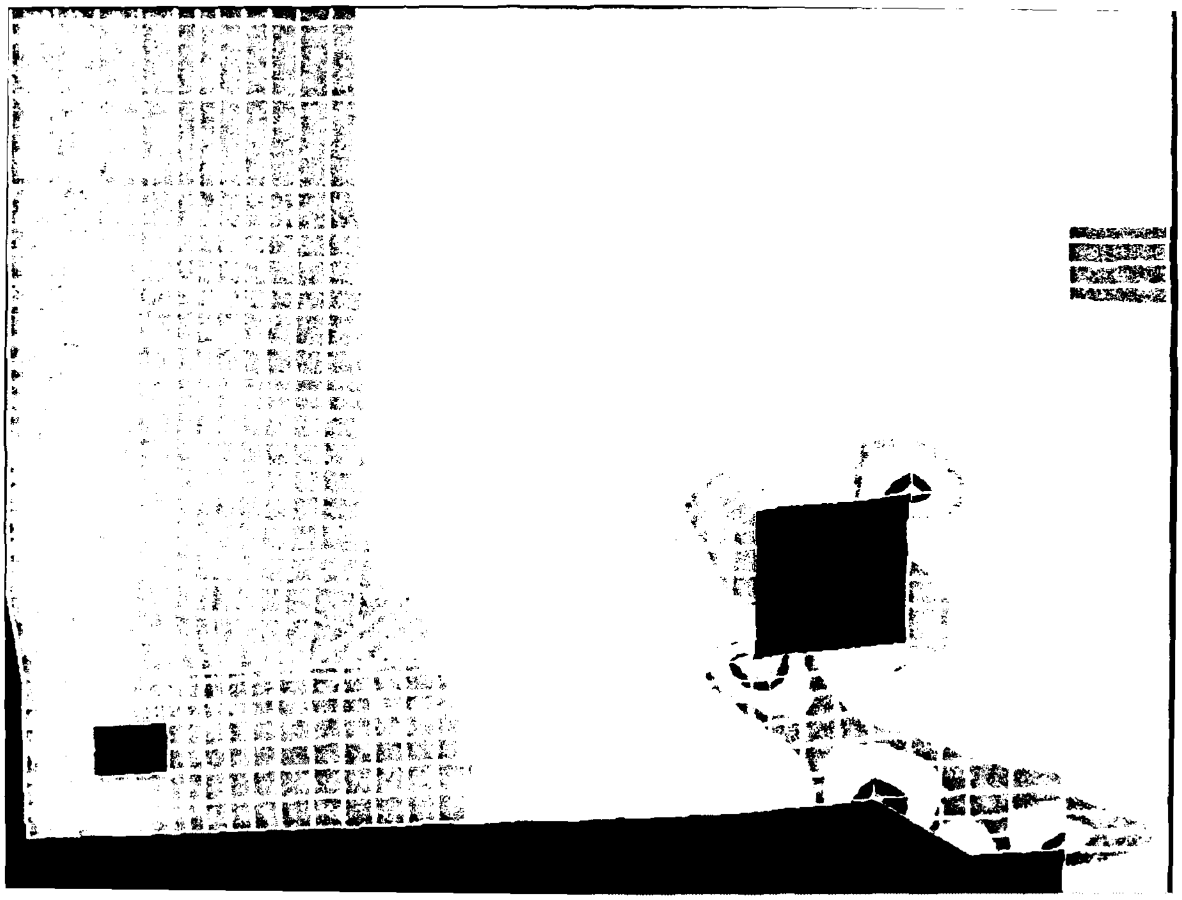


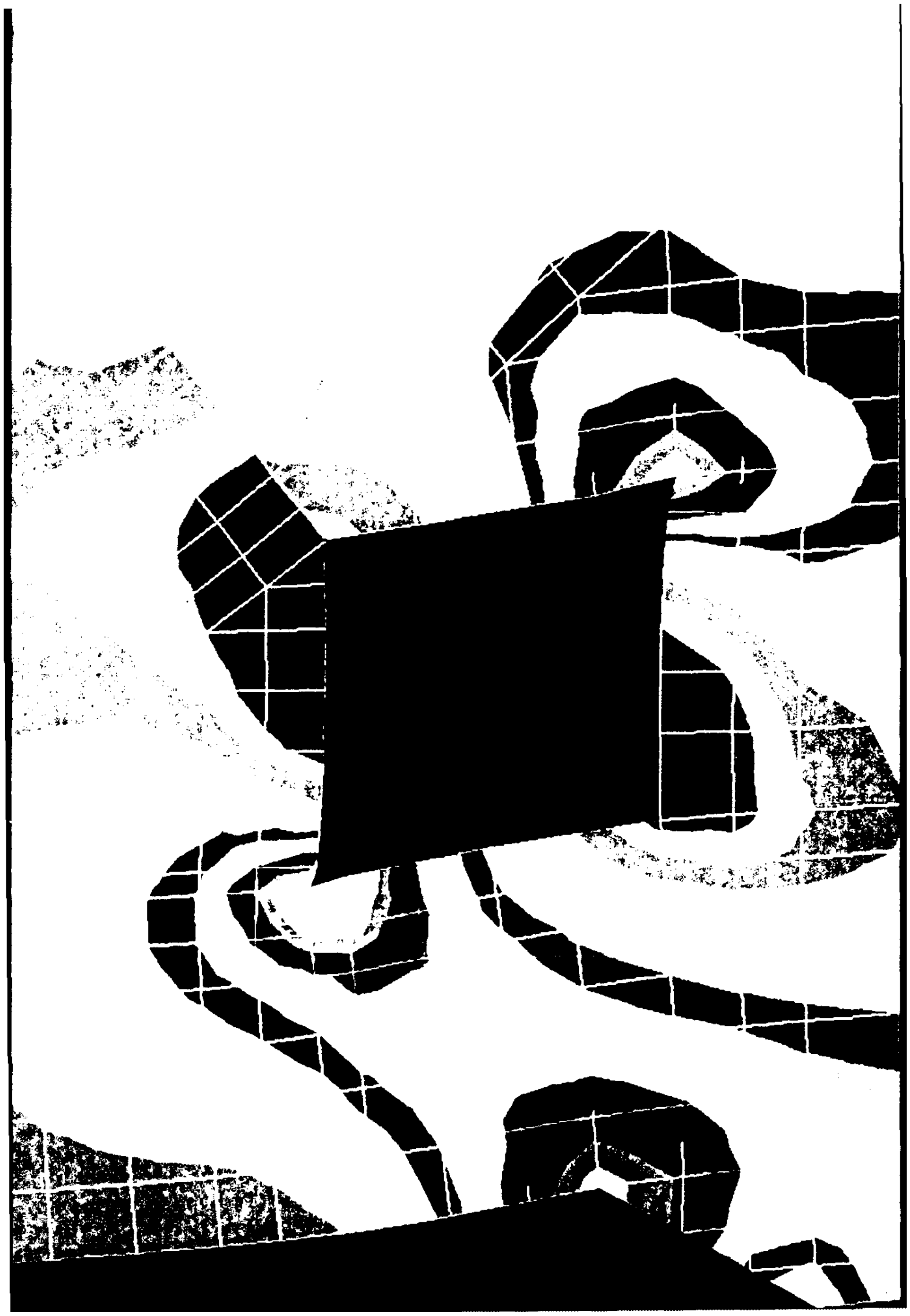


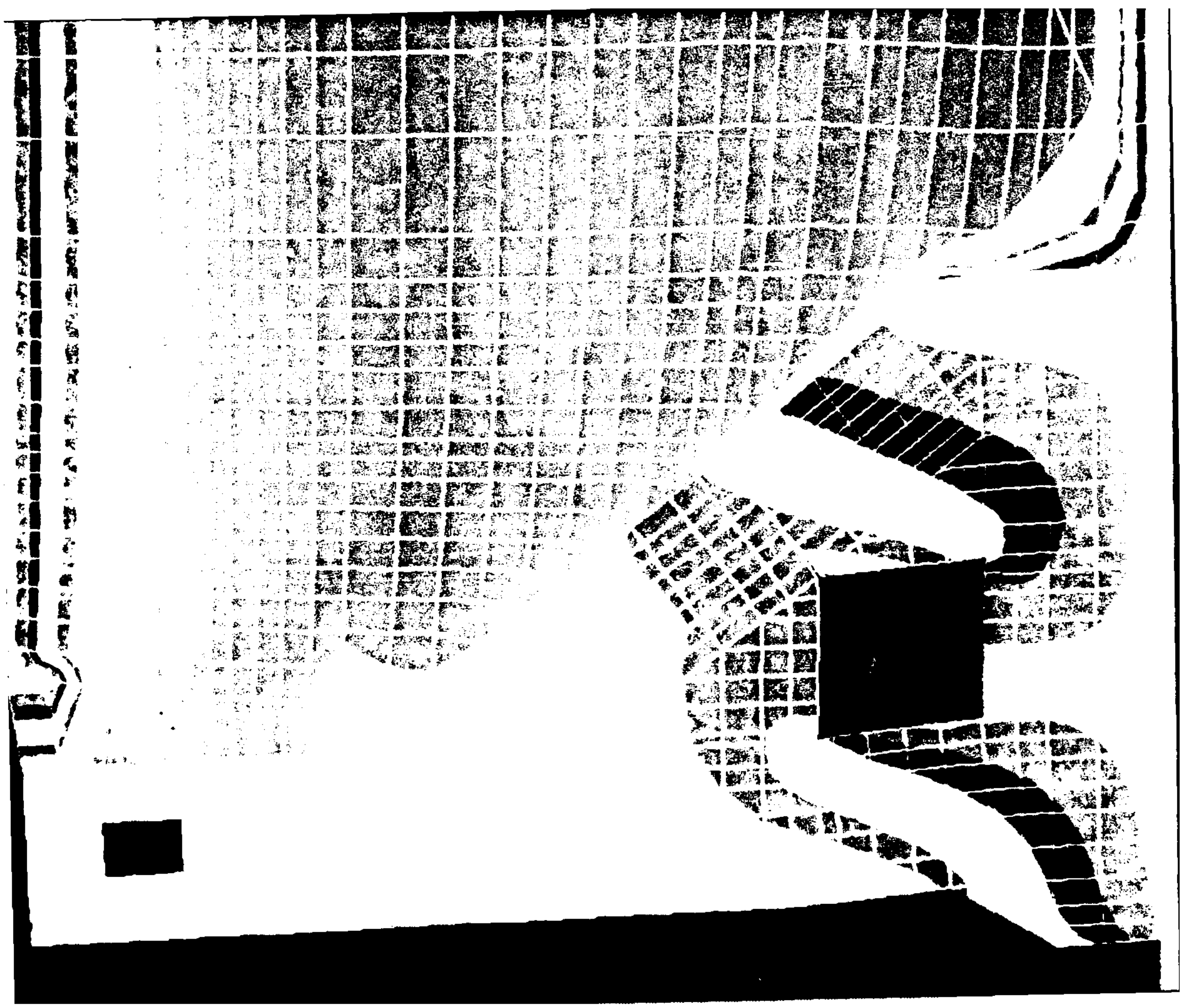


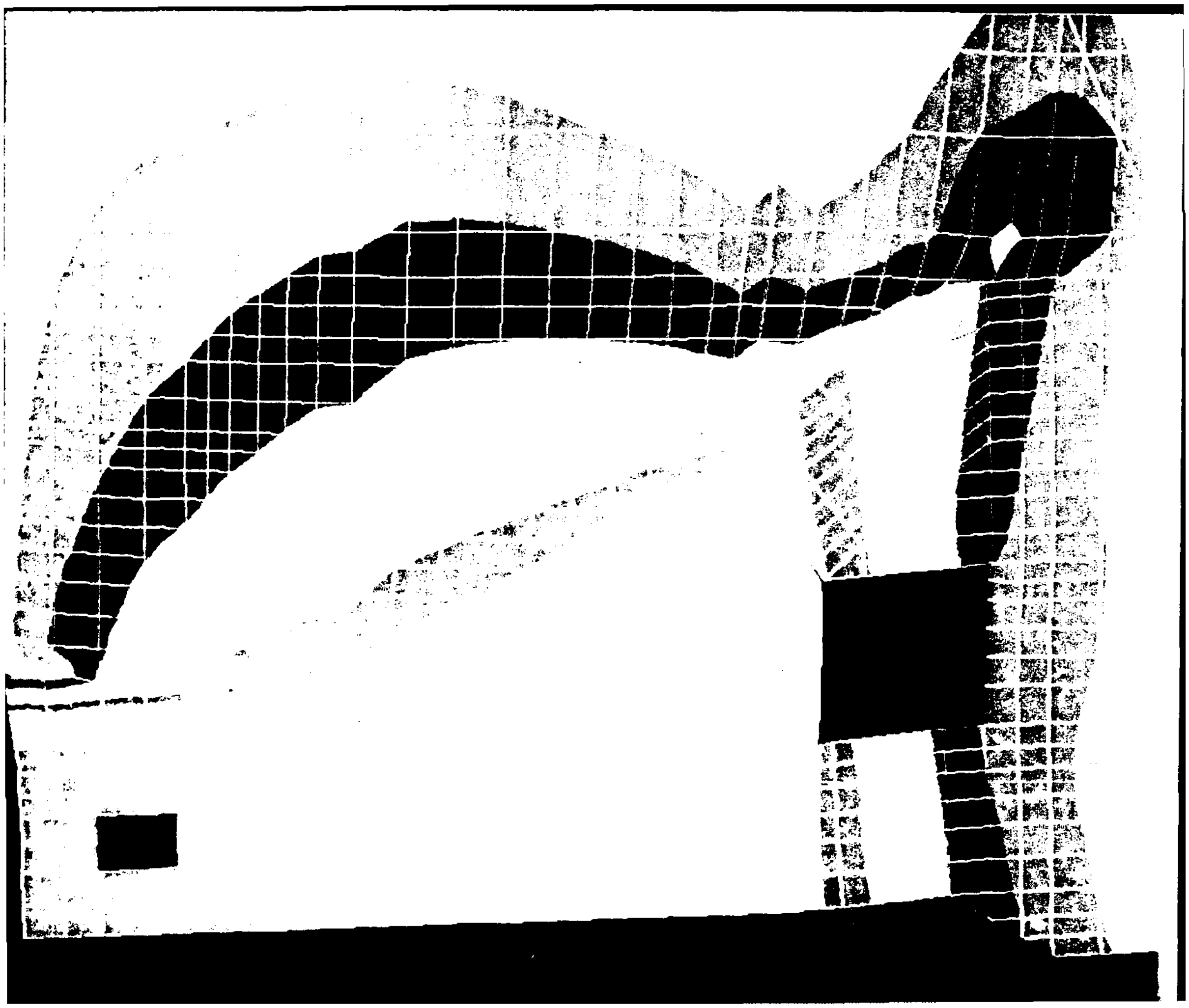


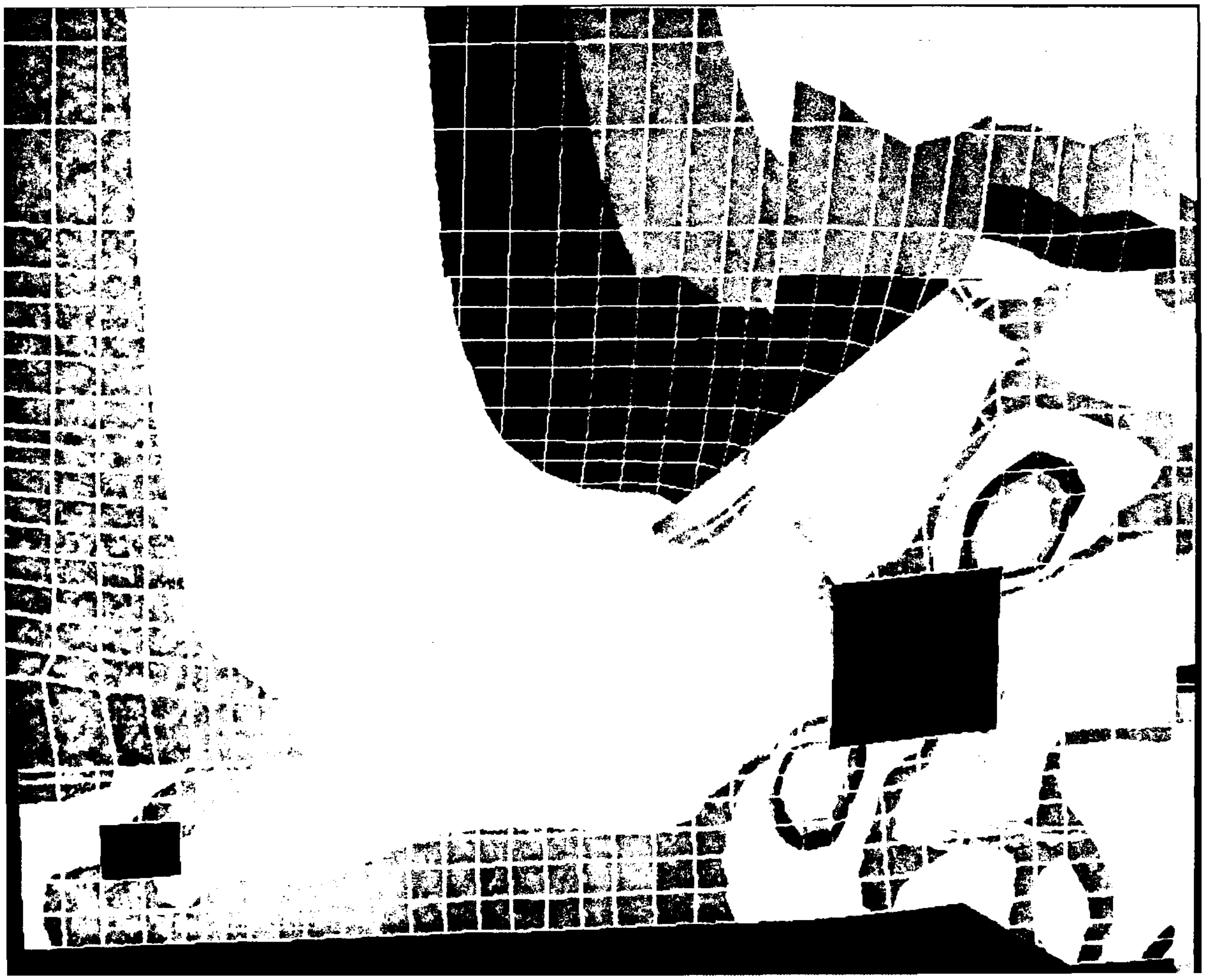


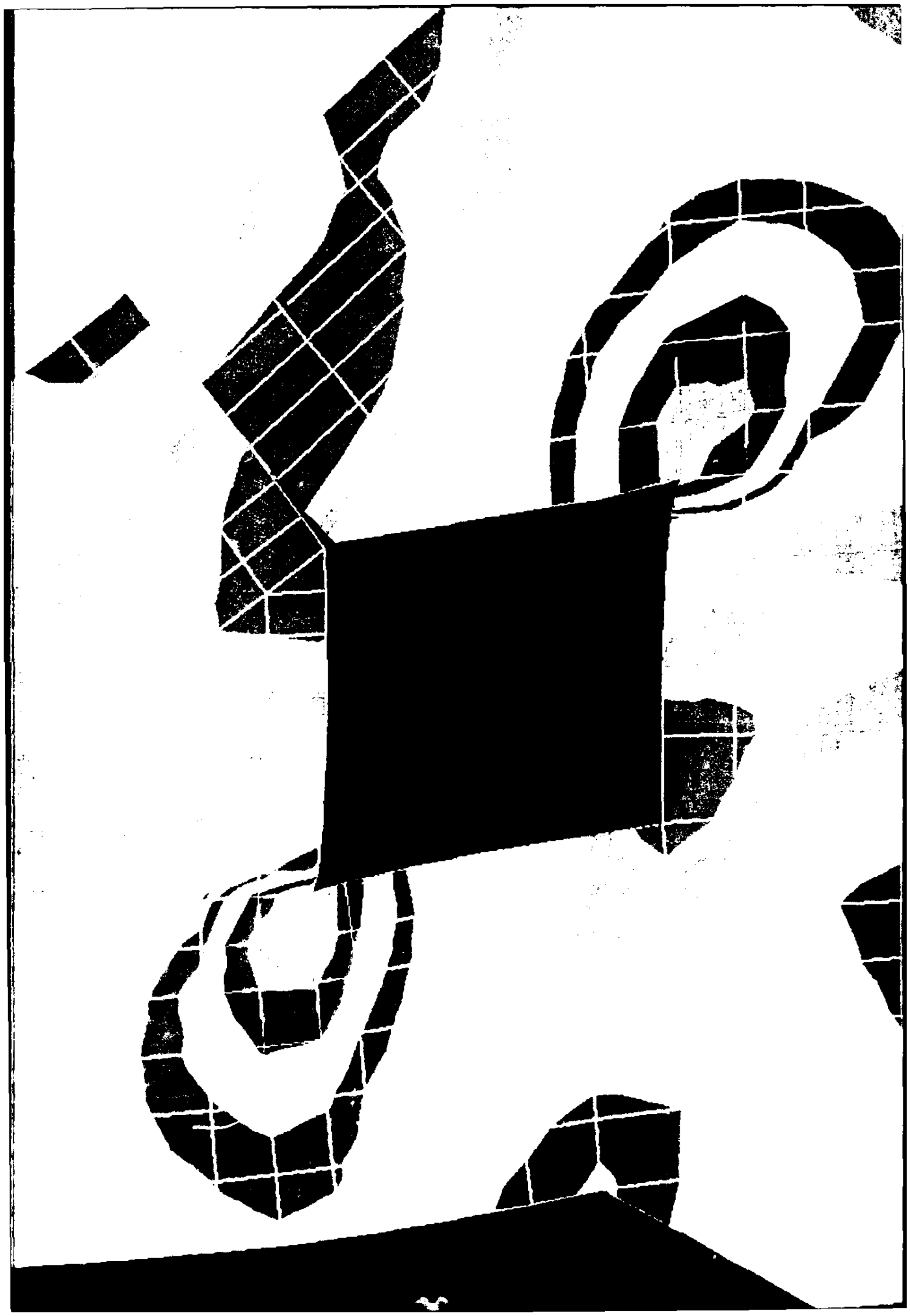


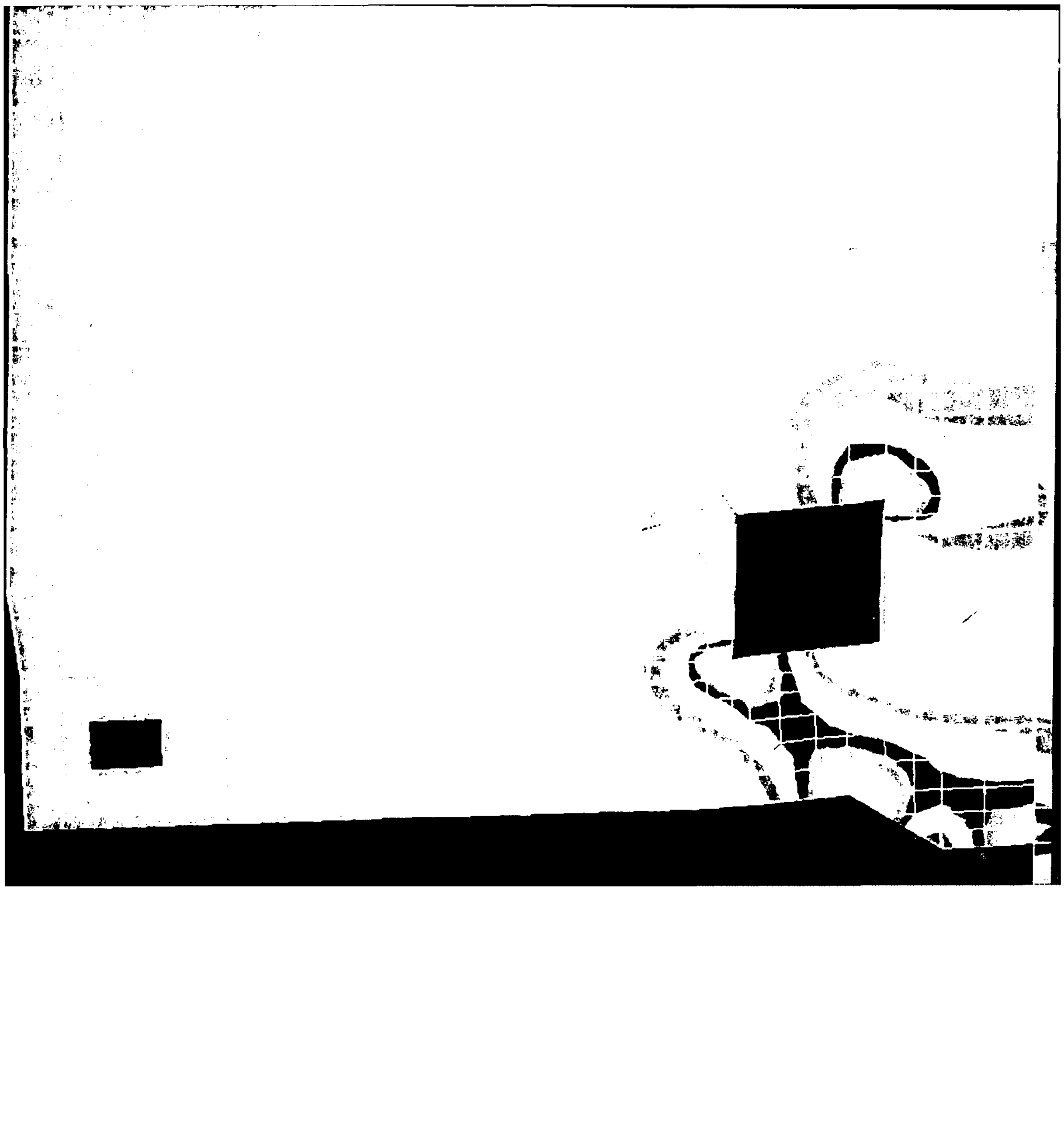


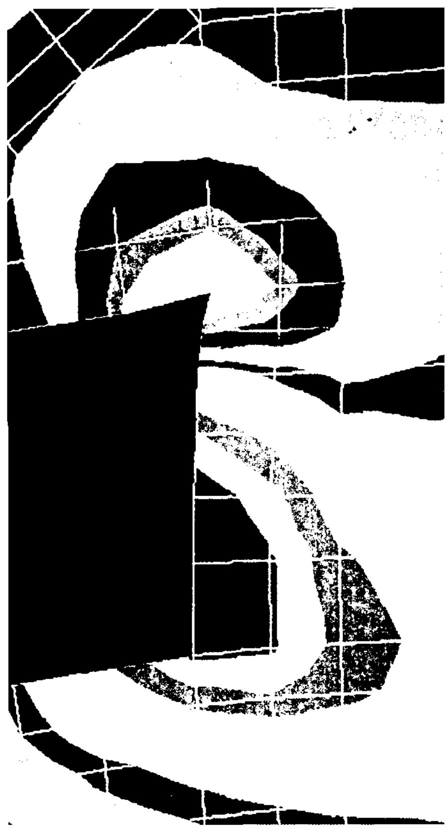


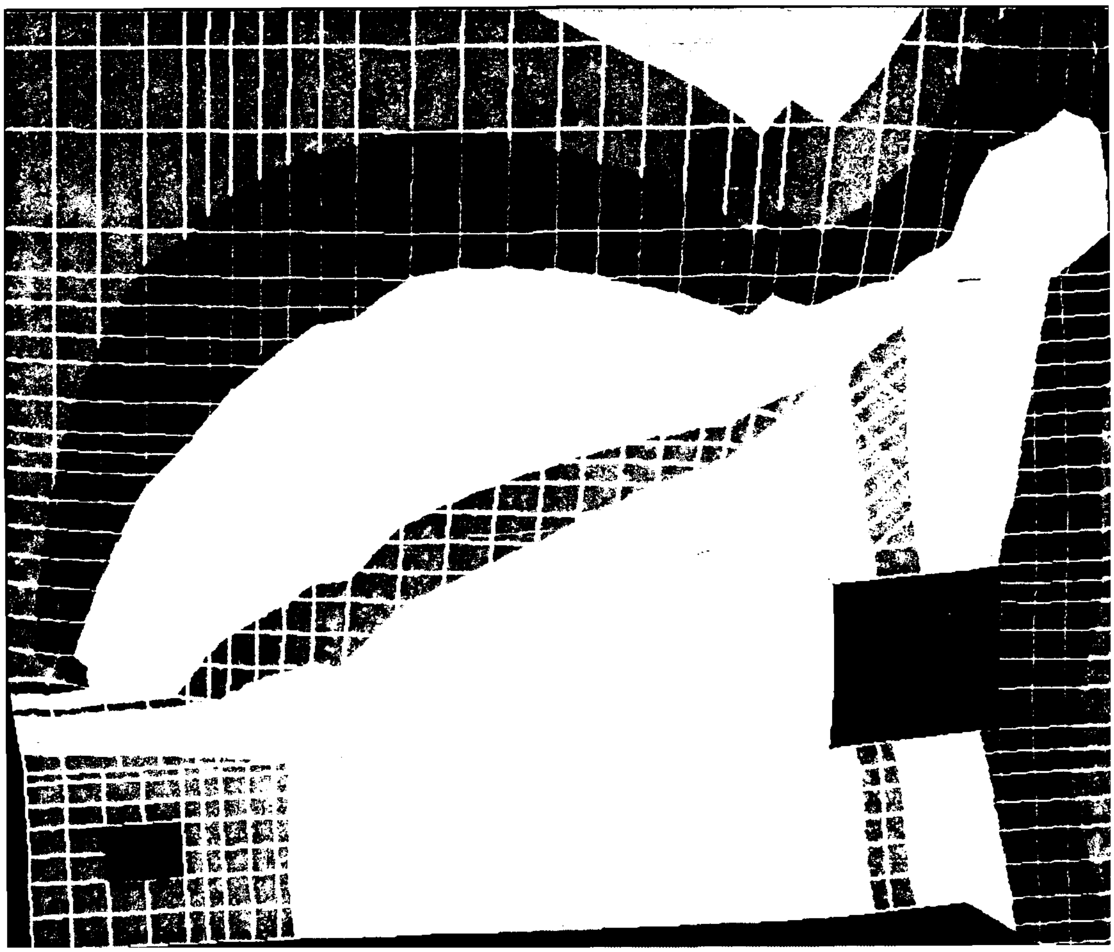


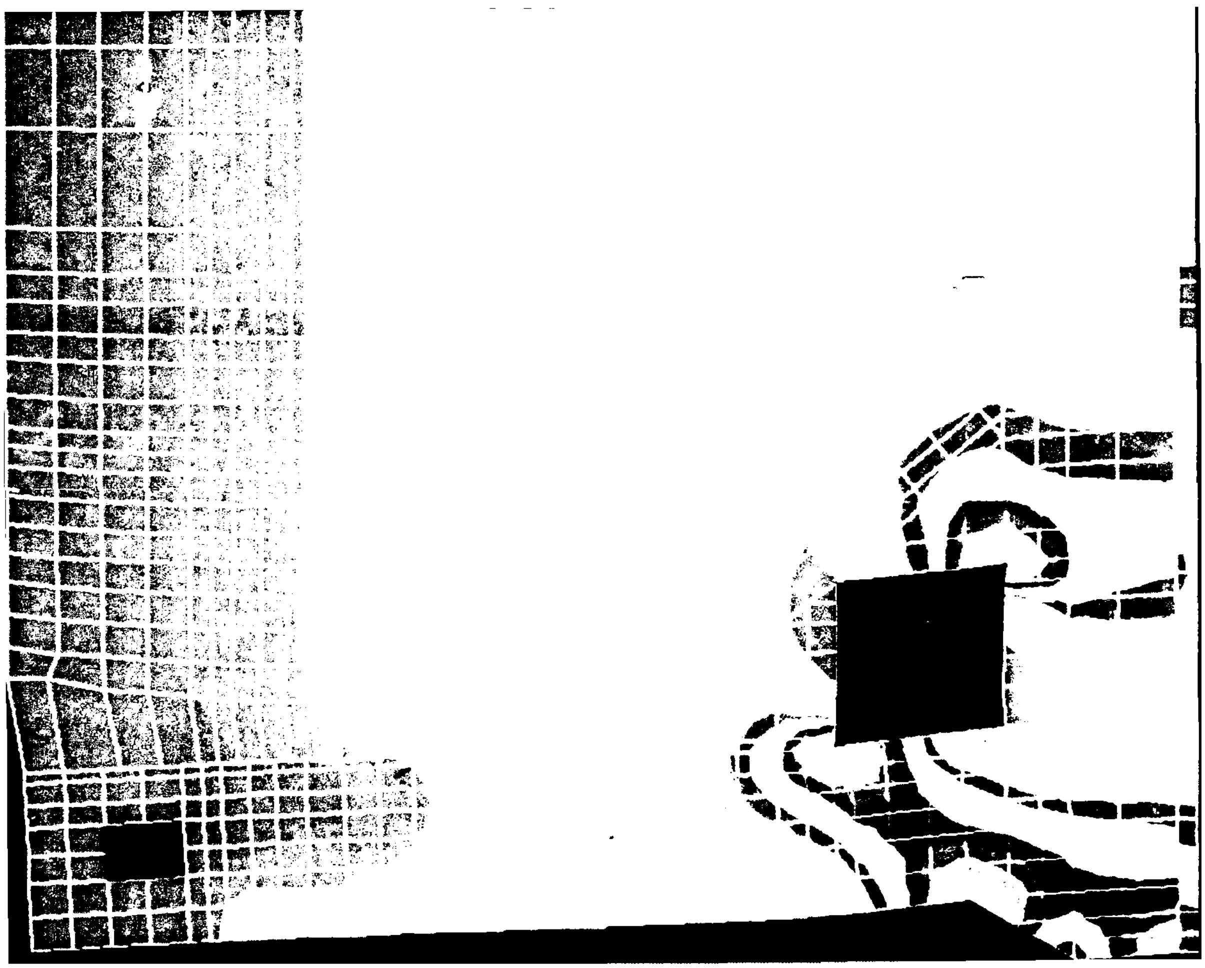


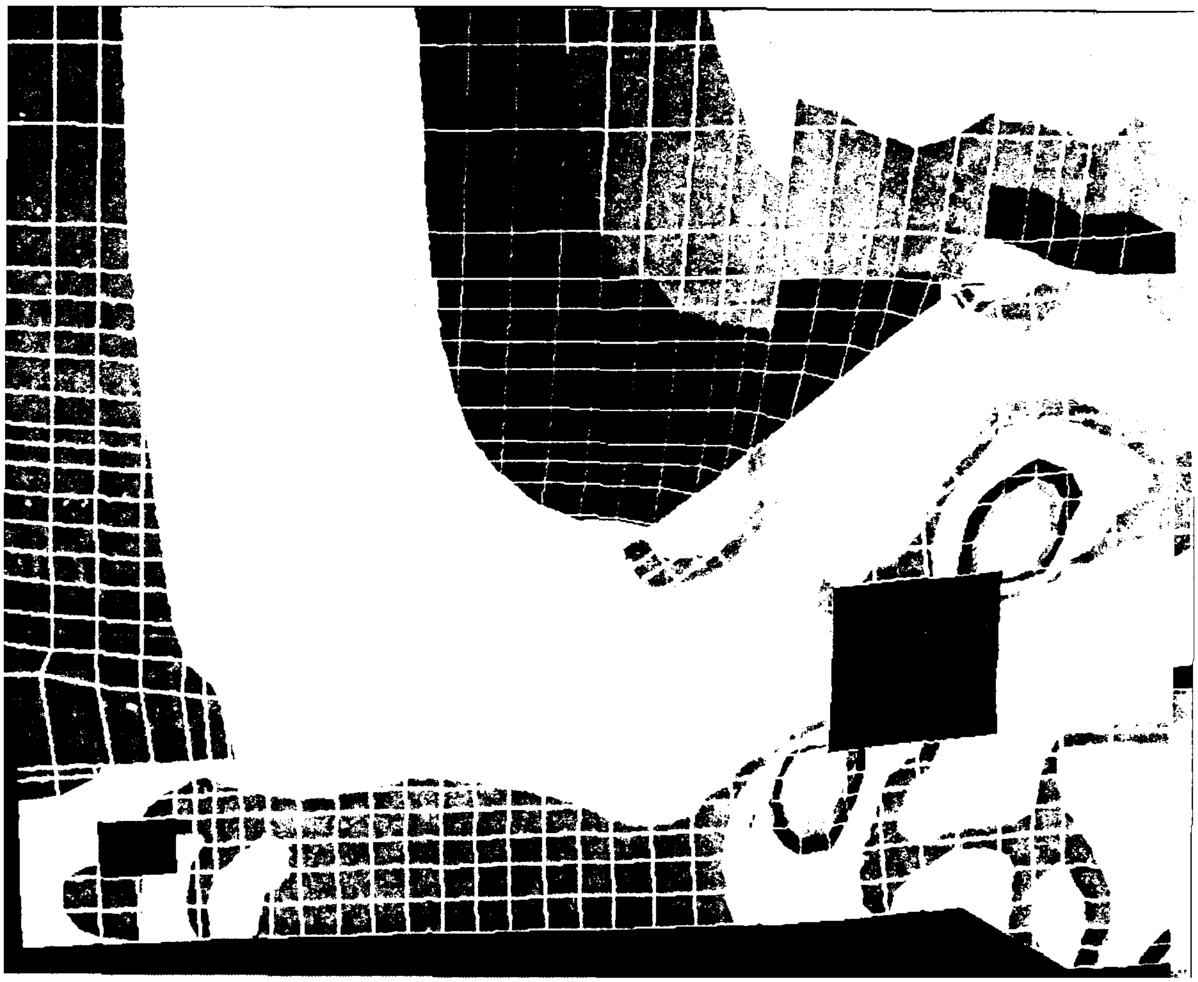


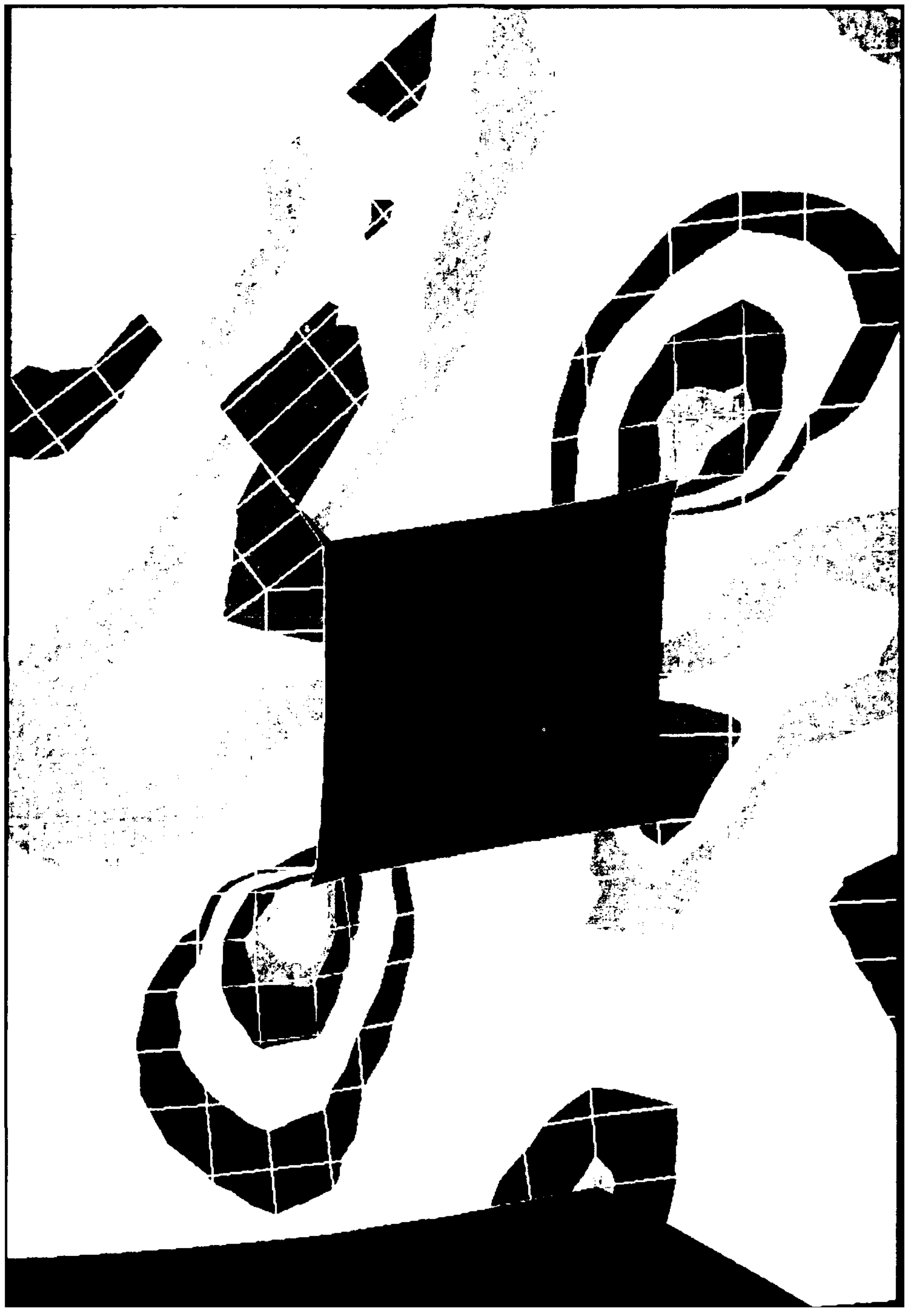


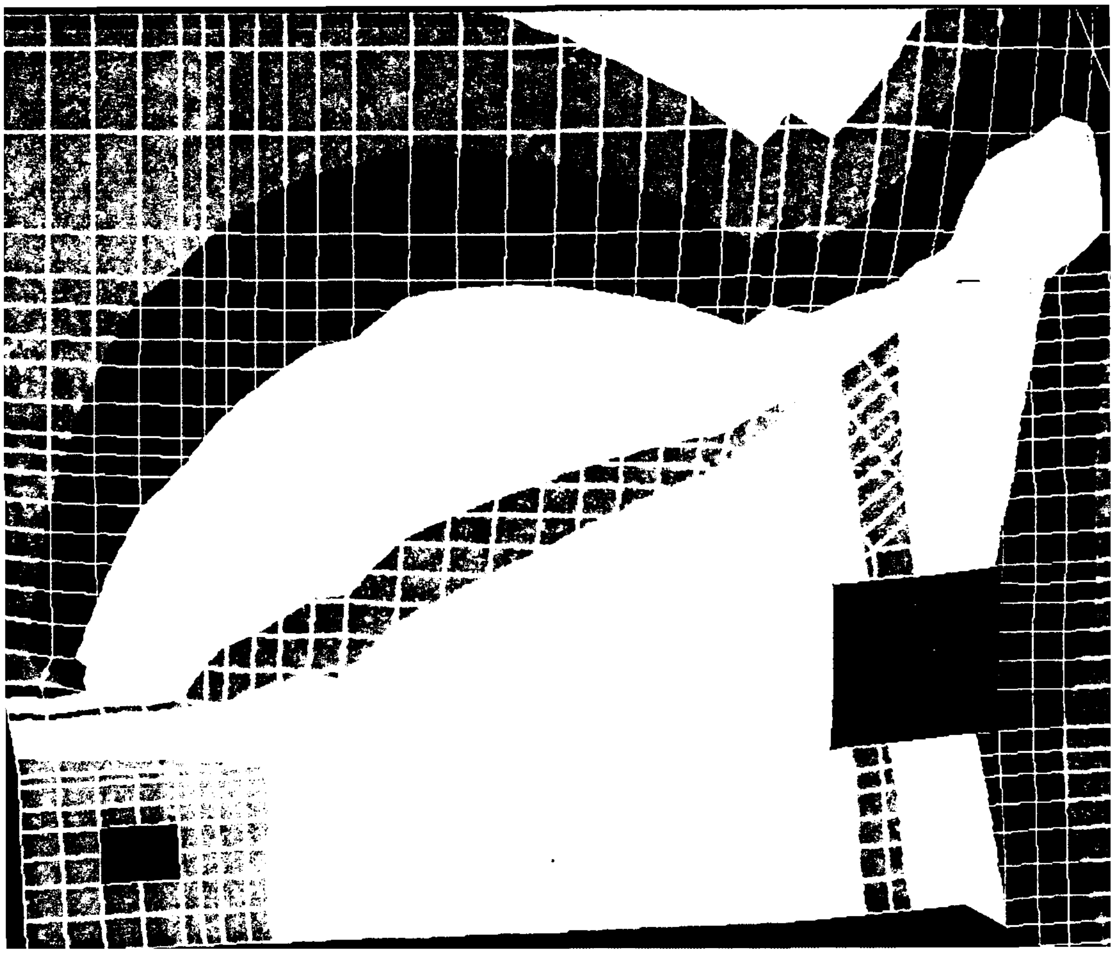


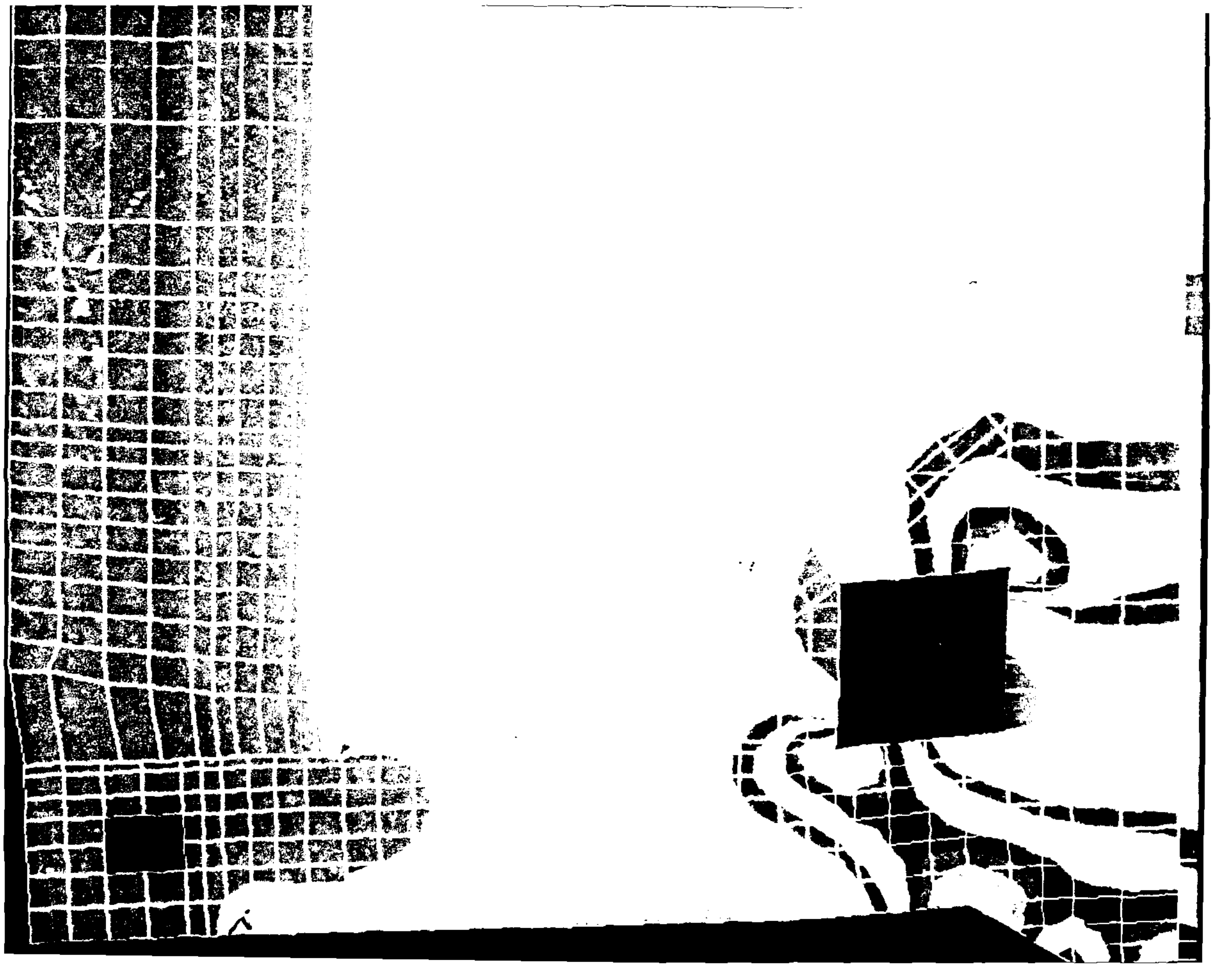


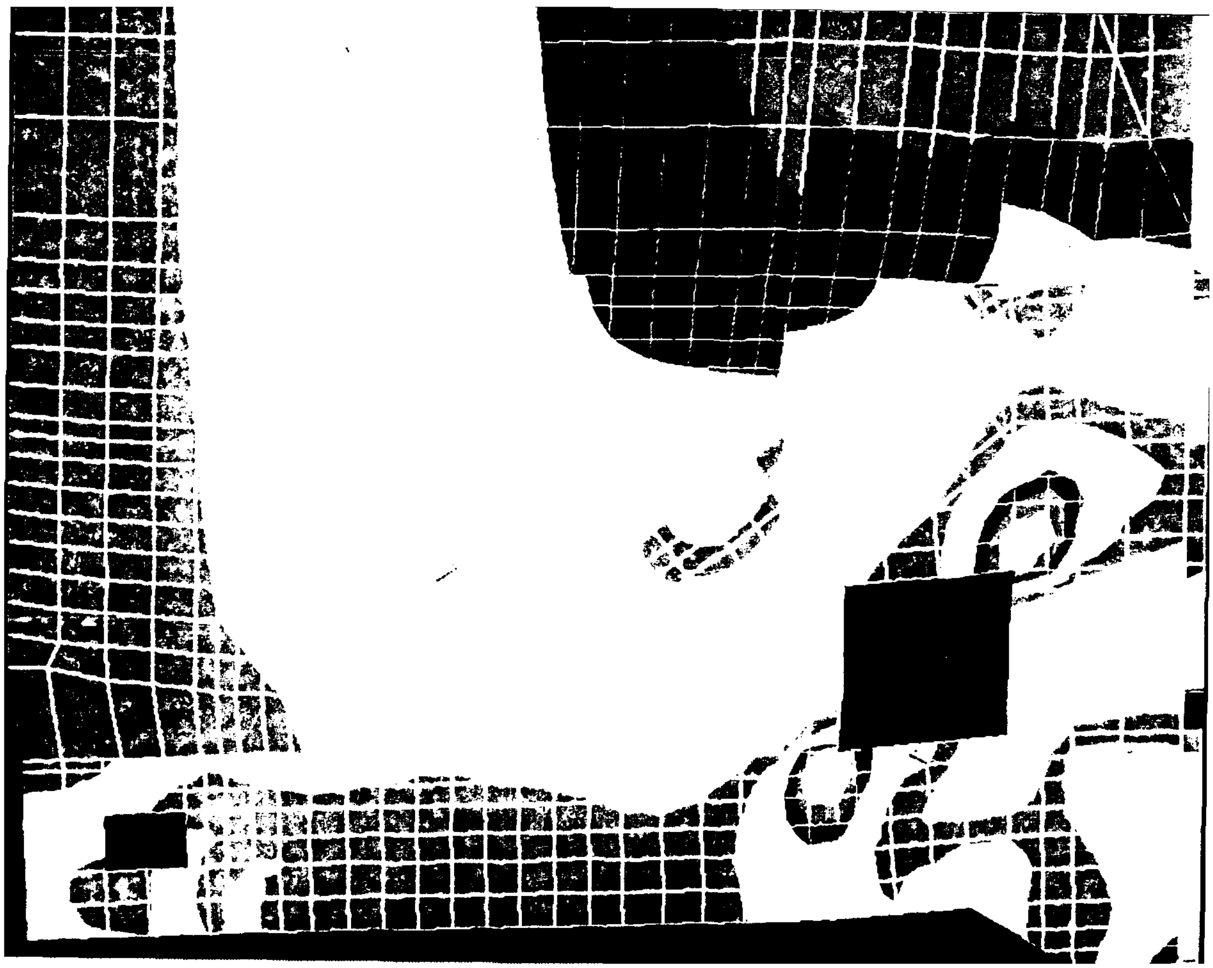


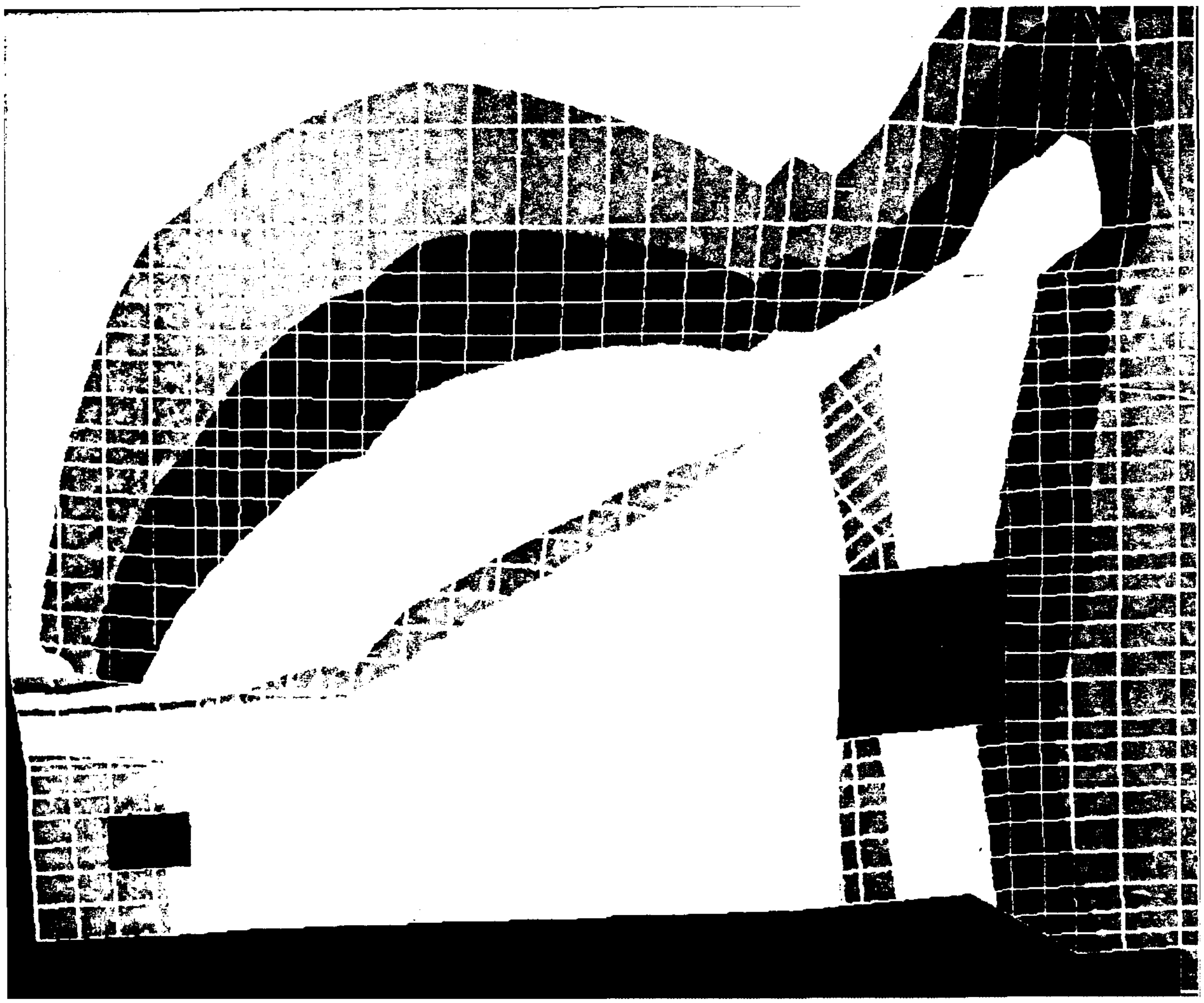


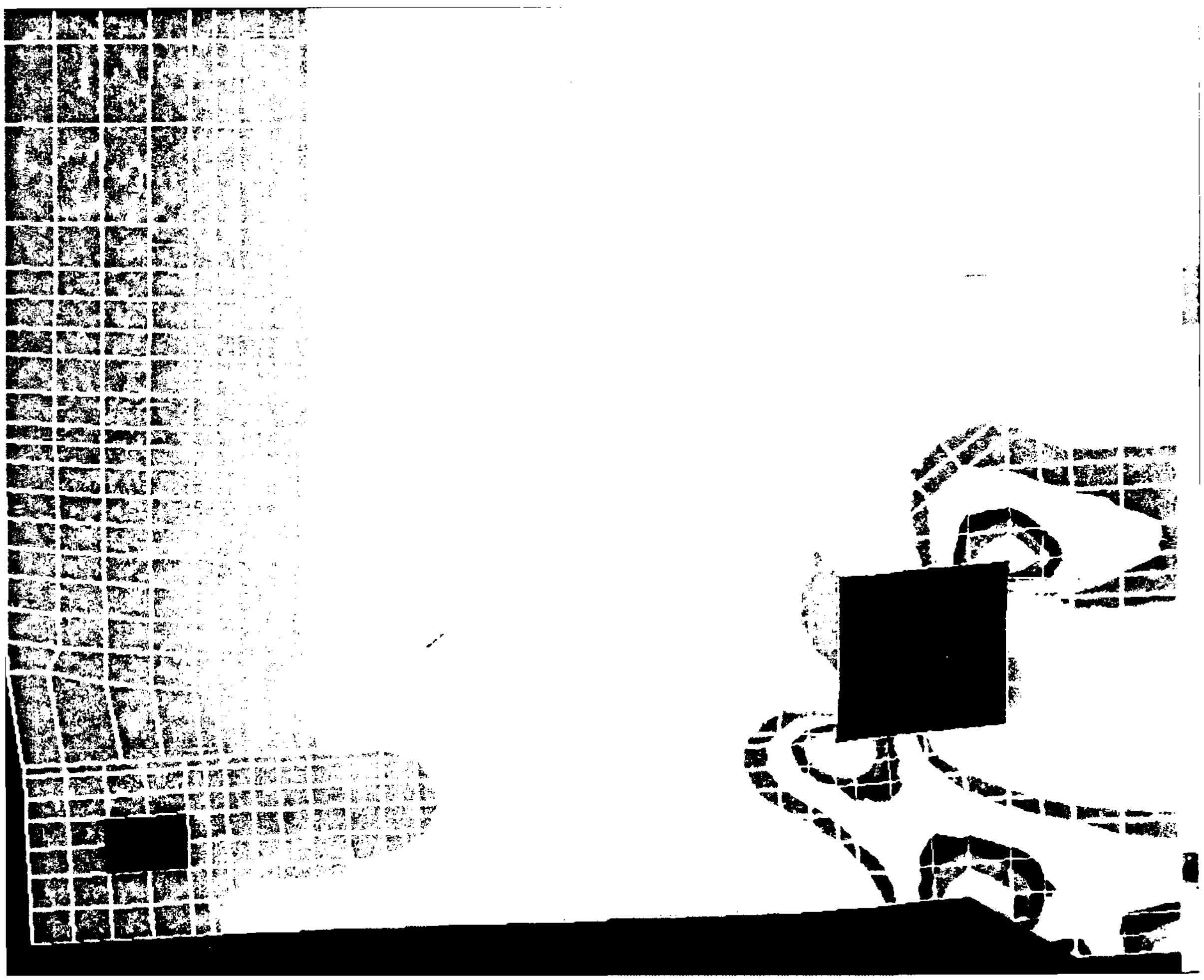


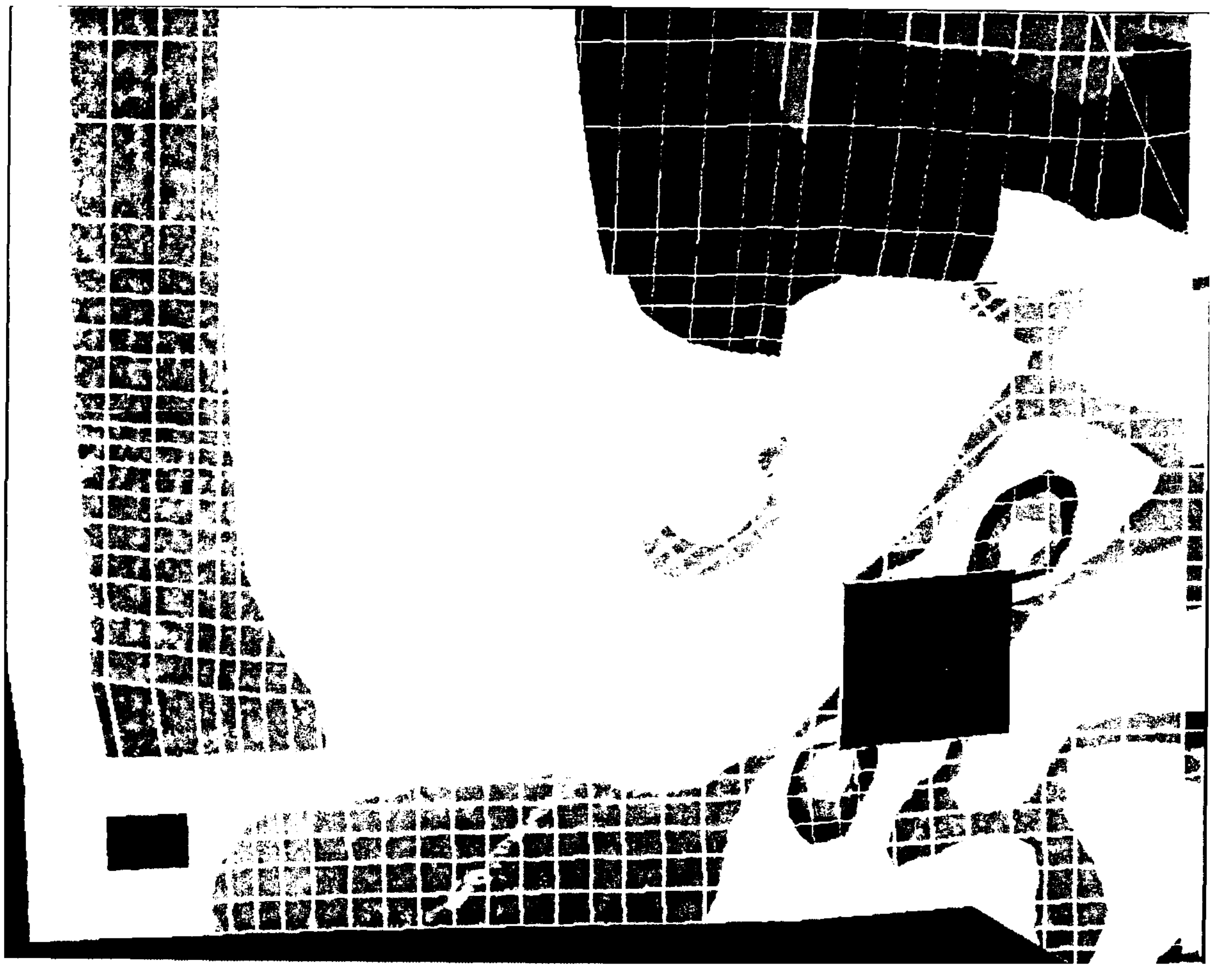












APPENDIX G: PETROGRAPHIC EXAMINATION RESULTS

U.S. Army Engineer Waterways Experiment Station 3909 Halls Ferry Road Vicksburg, Mississippi 39180-6199	Eisenhower & Snell Locks October 20, 1991 G. Sam Wong

Background

1. Cores from both Eisenhower and Snell Locks were taken for evaluation and examination. Portions of two cores from Eisenhower and one from Snell were selected for petrographic examination. When constructed in the mid 50's the two structure have been subjected to similar environmental conditions but severe deterioration was noticed in the concrete of Eisenhower Lock and was investigated by WES and reported by Buck in 1967, "Investigation of Concrete in Eisenhower and Snell Locks, St. Lawrence Seaway", TR No. 6-784. The current investigation also compares the concrete from the two structure.

Samples

2. Identifying data for the samples are shown below:

Serial No.	Lock Structure	Description
N-53-26-46	Eisenhower	North side of lock, Boring No. 53, Box 26, specimen #46, core run 104.8-108.6 ft, just above bedrock, 3610 psi
S-17-7-14	Eisenhower	South side of lock Boring No. 17, Box 7, specimen #14, core run 25.1-29.4 ft, upper portion of lock, 5980 psi
SN-S-15-1-50	Snell	South side of lock, Boring No. 15, Box 1, core #50, core run 0-4.5 ft, upper portion of lock, 7420 psi

Test Procedure

3. The as-received pieces of core were examined and subsamples were cut longitudinally and surface ground prior to examination.

a. Pieces were examined using a setereomicroscope.

b. Air content was determined using automatic image analysis system. The air voids in the sample were enhanced using a mixture of TiO_2 and grease to fill the voids.

Results and Discussion

4. Inspection of the three pieces of concrete indicated the following:

a. All pieces of core were similar in appearance. The maximum aggregate size was 3 in., 1.5 in., and 6 in. for samples N-53-26-46, S-17-7-14, and SN-S-15-1-50 respectively.

b. All of the broken surfaces appeared to represent fresh fractures. The breaks tended to go around the larger coarse aggregates and through the smaller coarse aggregates. Examination of freshly broken surfaces made during the examination also indicated similar features.

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c. The concrete was air-entrained. The air content for the samples examined ranged from 2.3 to 2.8 percent. Figures 1, 2, and 3 shows good distribution of air voids in the paste portion. Figure No. 2 indicated some tendency for air voids to be near the aggregate/paste interface of some coarse aggregate particles. Because of the relative small area of sampling for the air content determination, the low air content of the concrete is not believed to be of any significance. Previous average air contents reported by Buck were near 3.5 percent for the two lock structures.

d. The concrete contains crushed dolomite coarse aggregates and crushed dolomite fine aggregates. Reaction rims were not found in any of the aggregate particles. No other deleterious reactions were observed in any of the pieces of cores examined.

e. Use of phenolphthalein indicator to enhance the paste portion for image analysis indicated no carbonation of the paste as the entire paste portion of the concrete turned a reddish color.

Summary

5. Examination of two pieces of cores from Eisenhower Lock and comparison with concrete from Snell Lock indicated that concrete from both structures are similar and free from damaging cement-aggregate reactions and other deleterious reaction. The concrete from the two structures are similar.

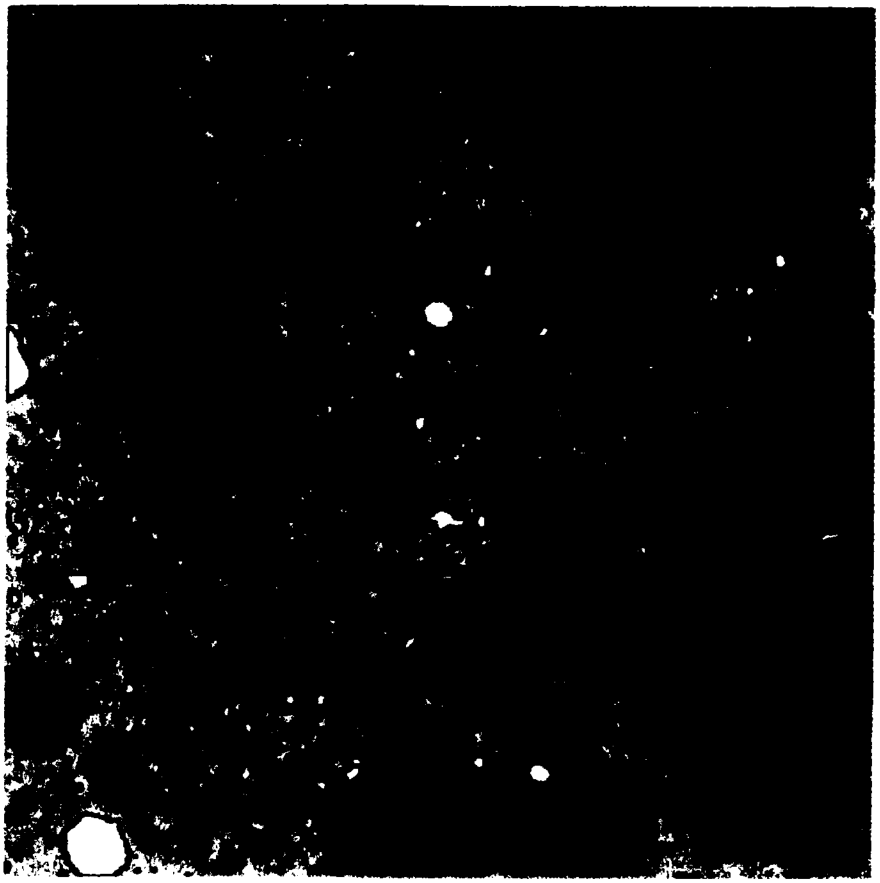

Figure No. 1. Sample N-5[illegible]-26-4c contains approximately [illegible] percent air. Air voids are circled in blue showing good distribution in paste. [illegible]X.

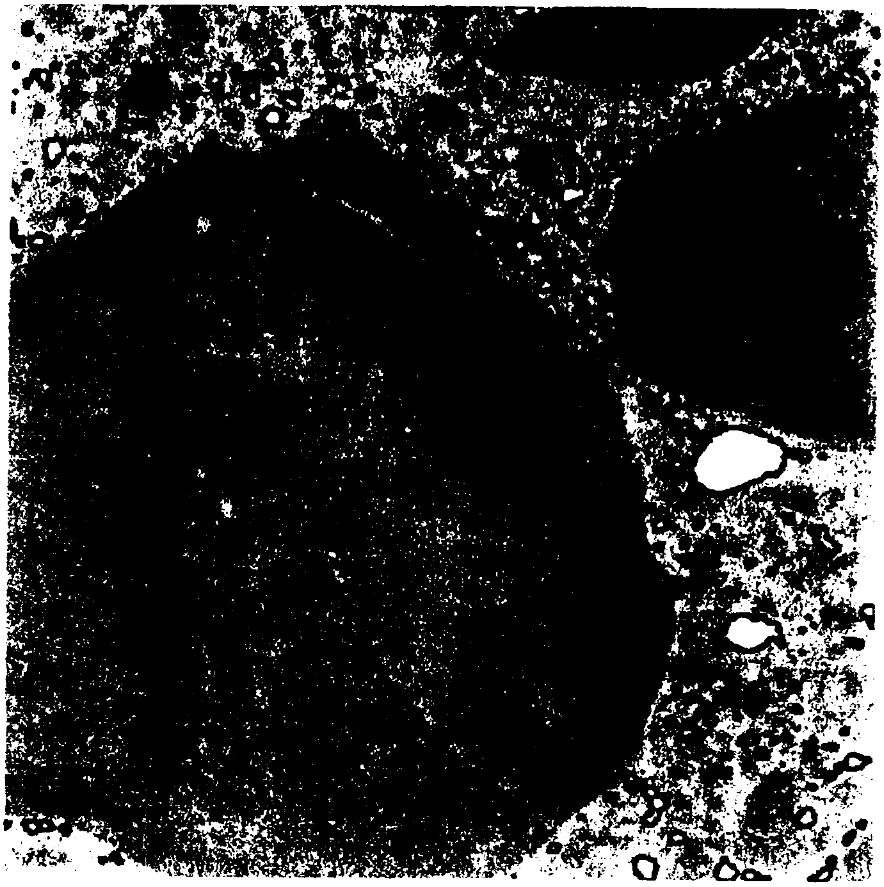

Figure No. 2. Sample S-17-[illegible] contains approximately 2.6 percent air voids. Some preferential alignment of air voids are noticed along large aggregate particles. 2X.

Figure No. 3. Sample SN-[illegible]-1-5[illegible] contained approximately 2.8 percent air. The air voids are evenly distributed throughout the paste. The dark spot on the large aggregate is ink used to cover large white crystals in the aggregate fraction of the concrete for image enhancement used during image analysis. 3X.